DEVELOPMENT OF GUIDELINES
FOR
PG BINDER SELECTION FOR WISCONSIN

WisDOT Highway Research Study 0092-01-01

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16. Abstract:

In 1997, the Wisconsin DOT made the transition to use PG binders in place of the Penetration and Viscosity graded asphalts that had been historically used. The decision at that time to specify the PG 58-28 as the standard grade for use was based on this grade similarity to the asphalts that had been used previously, the wide availability of the material in the region, and the fact that there was little or no difference in cost. Although this binder grade has been widely used, it is well known that the climatic conditions in the state of Wisconsin vary between the north region and south region of Wisconsin. According to the Superpave binder specification (AASHTO MP1) other grades designed for colder climate need to be used. In fact, before the AASHTO MP1 was implemented, more than one penetration grade was already being used in Wisconsin. In the cold regions of Wisconsin the Pen 200-300 grade was used while in the south the grades of Pen 85-100 and 120-150 were used.

Since the implementation of the Superpave binder specification in 1993 a large number of State Highway Agencies (SHAs) have found critical gaps in the PG grading system. Reviewing the published literature indicates that most of these gaps are somewhat related to the use of modified asphalts and to the fact that the existing PG grading system cannot discriminate between modification technologies, some of which are believed not be successful technologies in terms of adding value to the quality of binders.

Last year (2001) the NCHRP report 459 was published, which focused on offering a revised system for testing and evaluation of asphalt binders using a more mechanistic testing system based on direct measurements of damage behavior of binders. This report is offered to address the implementation of the NCHRP 9-10 proposal and to show how it could be implemented on trial basis using the results of testing of a wide selection of asphalts currently used, or marketed, in the State of Wisconsin. The binders include unmodified and modified of several grades (PG 58-28, PG 64-22, PG 64-34, PG 70-22, PG 70-28, and PG 76-28).

The results of testing using the NCHRP9-10 protocols were used to select binders for specific climate, traffic, and pavement conditions. The existing knowledge of PG grading of the binders was used to derive specification limits that would meet specific climate and traffic conditions. No grade shifting was necessary because all testing is done at actual pavement temperatures and because a more mechanistic approach could be used to meet the traffic volume and speed requirements.

The specification limits are tentative and mainly based on ranking of the PG grading system of the binders included. They serve as a starting point for future field validation and revision. This report is intended to describe the logic in deriving the limits and to offer initial specification limits to move us away from the Superpave plus and offer a logical alternative to compliment the existing PG grading system.

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EXECUTIVE SUMMARY

Project Summary

The result of this research is a proposal for improved guidelines for selecting asphalt binders. The guidelines consider new test procedures and specification limits that are applicable to both, modified and unmodified binders. They include proposed specifications for workability, rutting performance, fatigue performance and low temperature cracking resistance. The proposal was developed based on a large database of measurements collected for binders currently used in Wisconsin. Field validation is needed for the proposed guidelines in the future.

Background

In the United States, a number of State Highway Agencies claim that the Superpave specification has some critical gaps, mostly related to the performance characterization of modified binders. Recognizing this fact AASHTO sponsored project NCHRP 9-10 and in 2001 NCHRP 459 report was published. The report offered a revised system for testing and evaluating asphalt binders based on damage behavior. A scheme to conduct binder testing for rutting, fatigue, glass transition temperature, and workability, that would allow a more direct qualification of modified binders for specific climate and traffic conditions was presented. The proposal, however, was only conceptual and lacked the details required for implementation, such as specific criteria and limits. The purpose of this research is to address the implementation of the system for testing and evaluating asphalt binders in Wisconsin. The project was carried out by the Department of Civil and Environmental Engineering at the University of Wisconsin Madison through the Wisconsin Highway Research Program. The research team included Hussain U. Bahia (Professor and principal investigator), Kitae Nam (Graduate Research Assistant) and Rodrigo Delgadillo (Graduate Research Assistant). The project committee included Leonard Makowsky (chair of the Flexible Pavements TOC), and Mr.Tom Brokaw, from the DOT Traux Materials Laboratory .

Process

The study was conducted in three phases. In the first phase, necessary information as climate, traffic, and types of binders used in Wisconsin were collected. Also a review of the existing guidelines to select performance-graded (PG) binders for use in Wisconsin based on pavement temperature conditions, traffic speed, traffic volume, and pavement structure was conducted. In the second phase, which was a major part of the project, the laboratory testing was conducted to evaluate the effect of modification on binders at testing condition that simulate and consider various field condition, such as pavement and traffic conditions. The testing followed the Superpave binder testing protocols and their modification as proposed in the NCHRP 9-10 project report. The binder testing plan included one base asphalt and 18 modified binders. The testing procedure includes using the DSR for fatigue and rutting evaluation at high and intermediate temperatures; BBR, DTT, and GTT for the low-temperature cracking investigation and RV for the workability measurements.

As for the last phase of the project, specification limits that would meet specific climate and traffic conditions were derived. The limits are proposed as an outset point for further field validation. The analysis presented in this report includes the comparison of all tested binders and the development of specifications using the response variables measured during binder testing. The report is organized in the following chapters:

- Introduction
- Screening Testing and Workability Evaluation
- Fatigue Evaluation
- Rutting Evaluation
- Low-Temperature Analysis
- Summary of Specification Criteria
- Future Study

In each section the various testing procedures, criteria, and equipment that were used to compare all the tested binders are presented along with the data analysis.

Finding and Conclusions

The results of this project could allow a better characterization of both, modified and unmodified binders. The proposed testing allows appropriate ranking and selection of binders based on their performance with respect to fatigue, rutting and low temperature resistance. It also allows finding the proper conditions for mixing and compaction of mixtures produced with modified binders, evaluating the presence of particulate additives, and studying the storage stability of binders. A more detailed explanation of the findings is provided in the following points:

- For rutting evaluation of binders, traffic speed and traffic volume are considered in the specification in a more direct way. This allows to change the current procedure in which those important conditions are taken into account by shifting the high temperature grade, what is called “grade bumping”. The new proposed binder test (called creep and recovery) allows to evaluate the elastic recovery of the deformation which can be significant for modified binders.
- The fatigue evaluation and specifications include the pavement structure, traffic speed, and traffic volume, as conditions for selecting binders. Also two seasons, normal season and thaw season, are considered in the specification. The inclusion of two seasons allows direct evaluation of the influence of the temperature and the loss of support during the thaw season on the fatigue life of binders.
- The procedure to find the proper mixing compaction temperatures for all modified and unmodified binders considers shear rate effects. This is major improvement specially for the very viscous modified binders, where the current procedure resulted usually in the selection of excessively high working temperatures.
- The results indicate that the Particulate Additive Test (PAT) is a useful method to evaluate the presence of particulates in modified binders. The PAT test should be included in future binder specifications used in Wisconsin.
- Recommendations are presented to evaluate the storage stability of binders using a test called Laboratory Asphalt Stability Test (LAST). This test is complex and requires a long time. It is, therefore, not recommended to be included in future

specifications. It is, however, recommended that asphalt suppliers perform this test to evaluate the possible separation of modifier and binder during storage.

- Based on traffic and pavement information gathered a set of binder selection guidelines were developed. The guidelines include pavement temperatures, traffic conditions and pavement structure. The results of testing the 19 binders clearly show that the binders currently used in Wisconsin can fit in these guidelines. The selection criteria, however, need to be changed to include direct consideration of these important factors.

Recommendations for Future Action

The results of this research include initial specification limits that could serve as a base for further field validation. Therefore, the main future action required before the eventual implementation of these tentative guidelines is field validation. The validation should be focused on the specifications of binders during actual field service and workability during construction. For workability, the new mixing and compaction temperatures should be tried in real projects and the results should be evaluated in terms of adequate mixing (coating) and proper compaction to obtain the required densities. For the performance specifications (rutting, fatigue, and low temperature cracking resistance), field validation should include the selection of projects that cover the widest range possible of conditions. Samples of binder and mixture should be taken from the project and tested for the proposed parameters. A follow up of the performance of the projects for some years should be assured in order to correlate the testing results with the performance on site.

TABLE OF CONTENTS

DISCLAIMER I

ACKNOWLEDGEMENTS II

TECHNICAL REPORT DOCUMENTATION PAGE III

EXECUTIVE SUMMARY IV

TABLE OF CONTENTS VIII

LIST OF FIGURES X

LIST OF TABLES XI

LIST OF TABLES XI

CHAPTER 1: INTRODUCTION 1

1.1 Background and Problem Statement 1
1.2 Climatic Conditions in Wisconsin 1
1.3 Traffic Information 3
1.4 Review of Binder Selection Guidelines 3

CHAPTER 2: PARTICULATE ADDITIVES, STORAGE STABILITY, AND WORKABILITY 6

2.1 Particulate Additive Test (PAT) 6
Test Results 7
2.2 Laboratory Asphalt Stability Test (LAST) 8
Test Results 9
2.3 Workability of Binders 10
Test Results 11
2.4 Development of Zero Shear Viscosity (ZSV) Guidelines 12
2.5 Summary of Findings for Workability 17

CHAPTER 3: BINDER RUTTING EVALUATION 18

3.1 Background 18
3.2 Problems with the Superpave Rutting Parameter 18
Fully Reversed Load........ 18
Total Energy Dissipated per Loading Cycle........ 19
Number of Loading Cycles........ 21
Traffic Speed and Volume........ 21
3.3 A New Binder Parameter for Rutting 22
Cyclic Creep Loading........ 22
Number of Loading Cycles........ 22
Determination of the Viscous Component of the Creep Stiffness G_v........ 24
Traffic Speed and Volume........ 25
3.4 Testing to Derive Limits for G_V 26
Test Information 26

Summary of Repeated Creep Results 26
3.5 Derivation of Specification Criteria **28**
Mechanistic Binder Specification Framework 28
Weather and Traffic Information 28
Deriving Field Conversion Factors 30
3.6 Proposed Binder Criteria for Rutting **31**
3.7 Ranking of Binders **33**
3.8 Summary of Findings for Rutting **33**

CHAPTER 4: FATIGUE EVALUATION OF BINDERS 35

4.1 Problems with Superpave FATIGUE Parameter **35**
Strain Controlled Conditions 35
Dissipated Energy per Loading Cycle (Wi) 36
Numbers of Loading Cycles 36
Pavement Structure 37
4.2 A New Binder Parameter for Fatigue **37**
Number of Loading Cycles 38
Independence from the Mode of Loading 40
Pavement Structure 41
4.3 Determination of the Number of Cycles to Failure, Np20 **41**
Fatigue Relationship 42
Energy Input 42
Testing Temperatures 43
Testing frequencies 44
4.4 Testing to Derive Limits for N_{P20} **44**
Test Information 44
Summary of Binder Fatigue Testing Results 45
4.5 Computation of Np20 for Each Traffic Speed **48**
Fast Traffic Speed 48
Slow Traffic Speed 49
4.6 Derivation of New Specification Criteria **55**
Mechanistic Binder Specification Framework 55
Traffic and Weather Information 55
Deriving Field Conversion Factors 56
4.7 Binder Criteria for Fatigue and Ranking of Binders **61**
4.8 Proposed Binder Specifications for Fatigue **64**
4.9 Summary of Findings for the Binder Fatigue Study **65**

CHAPTER 5: LOW-TEMPERATURE CRACKING 66

5.1 Background **66**
5.2 Theory and Experimental Plan **66**
5.3 Test Results **69**
5.4 Construction of Binder Selection Criterion for Low-Temperature Cracking **73**

CHAPTER 6: SUMMARY OF FINDINGS AND RECOMMENDED FUTURE WORK 76

6.1 Summary of Findings **76**
6.2 Recommended Future Work **78**

APPENDIX I: LAST TEST RESULTS 80

APPENDIX II: TEST PLANS FOR FIELD VALIDATION 84

LIST OF FIGURES

Figure 1.1 Wisconsin 98% Confidence Interval Map 2

Figure 1.2 PG Binder Selection Criteria (WisDOT 02/2000) 5

Figure 2.1 Estimated Compaction Temperatures Using a HSV of 280 cP 12

Figure 2.2 Estimated Compaction Temperatures Using a ZSV of 6.0 Pas 13

Figure 2.3 Comparison Between HSV=280 cP, ZSV=6000 cP, and ZSV=3000 cP 14

Figure 3.1 Permanent Deformation of the Binder, Fully Reversed Loading 19

Figure 3.2 Recovering Capacity of Binders under Creep Testing 21

Figure 3.3 Loading Pattern and Binder Strain under Repeated Creep Test 23

Figure 3.4 Behavior of Binders of same PG grade Under Repeated Creep Loading 24

Figure 3.5 G_v v/s Allowable ESALs, Original Binder, High Traffic Speed 31

Figure 4.1 G* vs. Loading Cycles for Strain Controlled Testing 37

Figure 4.2 Variation in the DER for Stress Controlled Testing 39

Figure 4.3 Variation in the DER for Strain Controlled Testing 40

Figure 4.4 Fatigue life versus Stress Level Used in Testing 41

Figure 4.5 Amplitude Sweep Test Data Plot for Typical Binder and Np20 vs. W_i 43

Figure 4.6 Np20 versus Allowable ESALs, Average Pavement Structure 61

Figure 5.1 Failure Stress and Strain Data and Master Curves for C5 Binder 67

Figure 5.2 Glass Transition Measurement of C5 Binder 68

Figure 5.3 Estimated Cracking Temperatures Sorted by Criteria 70

Figure 5.4 Comparison between the PG grades 71

Figure 5.5 Linear Relationships between Critical Temperature and PG Grade 74

LIST OF TABLES

Table 2.1 Summary of the PAT tests	8
Table 2.2: The Result of the LAST Tests for C5 Binder	10
Table 2.3 Results of the ZSV Tests	15
Table 3.1 Measured G_v Values at 58^{o}C and 1.0 Second Loading Time	27
Table 3.2 Example of a Mechanistic Binder Specification Framework for Rutting	29
Table 3.3 Traffic Levels and Applicable PG Grades for High Traffic Speed	29
Table 3.4 Proposed Binder Rutting Criteria and Ranking of Binders	32
Table 3.5 Tentative Specifications for Rutting	32
Table 4.1. Results of Fatigue Tests at 6°C and 15°C (part II)	47
Table 4.2 Selected Wi Values for Strong and Weak Structure, Fast Traffic	49
Table 4.3 Wi Values for Slow Traffic	50
Table 4.4 Selected Wi Values for Strong and Weak Structure, Slow Traffic	52
Table 4.5 Estimated Np20 Values for Fast and Slow Traffic (Part I)	53
Table 4.6 Example of a Mechanistic Binder Specification Framework for Fatigue	55
Table 4.7 Np_{20} Values for Average Pavement Structure	59
Table 4.8 Grouping of Binders and Traffic Levels for Fatigue Field Factors	60
Table 4.9 Np_{20} Values for Each Binder Group and Temperature	60
Table 4.10 Binder Criterion for Fatigue and Ranking of Binders	63
Table 4.11 Proposed Specifications for Fatigue Performance of Binders	64
Table 5.1 Results of the Low-Temperature Cracking Tests	69
Table 5.2 Summary of Glass Transition Measurements	72
Table 5.3 Applicability of Tested Binders for Low-Temperature Cracking	75

CHAPTER 1: INTRODUCTION

1.1 Background and Problem Statement

The Wisconsin Department of Transportation implemented the AASHTO MP1 binder specification for Performance graded (PG) binders in 1997. At that time the DOT technologists decided implementing the standard by selecting one binder grade (PG 58-28) as the standard grade for use throughout the state. The decision was based on similarity to asphalts used previously and the wide availability of the material in the region. This binder grade had resulted in pavements that performed relatively well for the past two years.

The PG grading system is designed to allow selecting different grades based on the extreme temperatures, traffic volume, and traffic speeds. To fully implement AASHTO MP1 and to make use of the grade system, guidelines for selecting the PG grade based on local climate, traffic conditions, and pavement structure are needed.

This important advantage of this system was not being used in the system of one PG grade in Wisconsin. After a few years of successful transition into the PG system, the DOT developed a more detailed system for selecting the grade based on climate and pavement conditions.

1.2 Climatic Conditions in Wisconsin

It is well recognized that there is more than one climatic region in Wisconsin that requires more than the PG58-28 grade. In particular it is know that he low temperature range varies significantly and required more grades for pavement temperatures lower than (-28). Figure 1.1, developed by Mr. Gerald Reinke of MTE in Wisconsin, shows three different climate regions in Wisconsin. These regions are divided by their cold temperatures.

As can be seen, three PG binders are needed in accordance with the cold temperatures of the regions. These binders are PG 58-28, PG 58-34, and PG 58-40. As a standard grade binder, PG 58-28 binder is used southeast areas of Wisconsin, PG 58-34 for most of southwest and central areas of the state, and PG 58-40 for the rest of the regions, mostly northern parts of Wisconsin.

In addition to the cold temperature regions, it appears that there are two hot temperature regions in this map that accept two PG binders; PG 52-xx and PG 58-xx. However, since a PG 58 grade would cover most areas of Wisconsin, using a single high PG grade (PG 58-xx) is expected to reduce complexity of selecting binders.

As discussed earlier, pavement temperature is only one factor in selecting performance grading. Traffic volume and speed is the second level of selection.

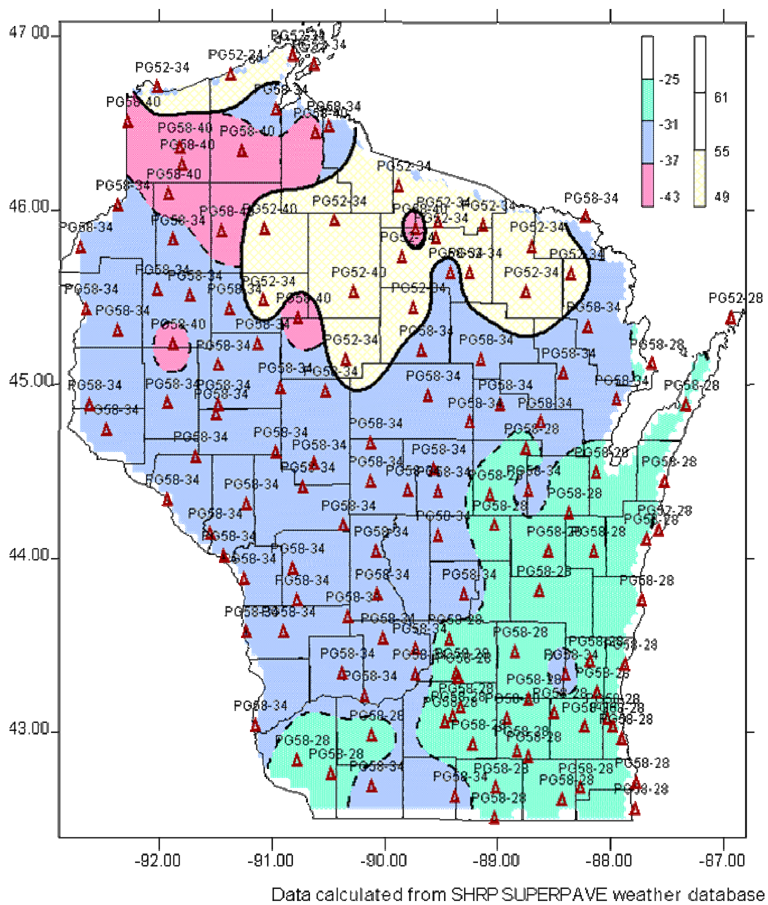


Figure 1.1 Wisconsin 98% Confidence Interval Map Based on Nov 1997 Revision of Pavement Temperature Algorithm

1.3 Traffic Information

It is essential to consider traffic condition for binder selection. Thus recent traffic data and information for Wisconsin have been collected in a form of a map based on information from the WisDOT Division of Transportation Investment Management, the U.S. DOT, and the Federal Highway Administration. In this map, the traffic information is based on data collected by the ATR (Automatic Traffic Recorder) recorded at stations spread throughout the state of Wisconsin. The data are shown in terms of AADT (Annual Average Daily Traffic) and is estimated from documents published in 1999 by WisDOT. The data were sorted into several necessary categories such that it can be effectively used to select the grade of binder based on traffic information. For example, each county throughout the state is assigned certain levels of traffic volume that will eventually be combined with pavement temperature information to specify required binder. Based on this approach, a county may include several different binder grades due to different levels of traffic volume.

This is not a completely new concept; WisDOT has published in 2002 a tentative PG Binder Selection Criteria, as shown in Figure 1.2. The selection guideline was developed to account for the effects of layer position, traffic speed and traffic volume based on grade shifting. This research has been intended to revise these criteria. The revision includes advanced methods of evaluating modified binders and will include specific procedures for considering such conditions as mentioned earlier just rather than grade shifting. It is expected that the final results of this research will allow shifting the criteria towards direct evaluation of traffic speed and volume rather than grade shifting based on specific guidelines.

1.4 Review of Binder Selection Guidelines

The tentative PG Binder Selection Criteria shown in Figure 1.2 suggests using more than one PG grade binder such as PG 64-28, PG 64-22, and PG 70-28 according to traffic conditions and pavement layer. For example, PG 64-28 is recommended for the areas of increased turning, stopping, parking movements, intersection of highways, or high traffic volume. Just as this PG binder is recommended for new base layers, so PG

64-22 binder is suggested for overlay use. PG 70-28 binder is only for areas of high traffic (more than 10 million ESALs).

Although different grades of binders rather than one standard binder are recommended in these criteria, recent research shows that grade shifting could be misleading because it is requiring testing at artificially high temperatures that pavements will not experience. For example using a PG 76 grade, tested at 76 C is not the best procedure because pavement will not experience the 76 C at any time. It is this better to continue testing at pavement design temperature, 58C for Wisconsin and use other more scientific methods to account for traffic volume, traffic speed, and pavement structure.

Modification of asphalts, when such a system is used, will focus on improving resistance to traffic volume or speed rather than on meeting an artificially high temperature requirement.

In the following sections, the details of the testing methods and the collected data and analysis are presented to confirm the idea of using standard PG binders in Wisconsin with a different method for accounting for traffic volume and speed.

PG Binder Selection Criteria
Wisconsin Department of Transportation
(02/21/2000)

I. Upper Layers:
   A. Rural Projects: (for ≥ 4 mill ESALs see high traffic volume)
      1. New Base: PG 58-28
      2. Overlay: PG 58-28

   B. Urban Projects & Sections: (area of increased turning, stopping, or parking movements; waysides, parking lots)
      1. New Base: PG 64-28
      2. Overlay: PG 64-22

   C. Stop Condition Intersections: (i.e. intersection of 2 US highways with turning movements)
      1. New Base: PG 64-28
      2. Overlay: PG 64-22

   D. High Traffic Volume:
      1. Sustained Speed < 55 mph
         a) ≥ 4 million ESALs: PG 64-28
         b) ≥ 10 million ESALs: PG 70-28
      2. Sustained Speed > 55 mph
         a) ≥ 10 million ESALs: PG 64-28

II. Lower Layer:
   A. PG 58-28: normal
   B. PG 64-22: if matches upper layer

Notes:

Prepared by Wisconsin Department of Transportation, Bureau of Highway Construction, Quality Management Section, Thomas F. Brokaw, Asphalt, Aggregate, & Soils Engineer

1. All ESAL designations are 20 year design life values.
2. Use only 2 different PG grades per project.
3. If you have any questions about these guidelines or their application, please contact:
   Thomas F. Brokaw, Asphalt, Aggregate & Soils Engineer
   Quality Management Section
   DTID, Bureau of Highway Construction

Figure 1.2 PG Binder Selection Criteria (WisDOT 02/2000)

CHAPTER 2: PARTICULATE ADDITIVES, STORAGE STABILITY, AND WORKABILITY

In the NCHRP 9-10 project it was concluded that there is a need for screening tests that evaluate presence of particulate additives, evaluate storage stability, and to estimate workability of binders for mixing and compaction. In this project, the type and amount of additive used in modification was measured using the PAT test. The Laboratory Storage Stability Test (LAST) was used for measuring the storage stability. The procedure recommended for estimating the Zero Shear Viscosity using the Rotational Viscometer was used to estimate the workability. The results were summarized and the main findings are discussed in the following sections.

2.1 Particulate Additive Test (PAT)

One of the alternatives to using microscopy to determine the nature of the asphalt additives is separation of the additive from asphalt. With separation, the general type of the additive and its characteristics can be determined. In a PAT, a diluted solution of the asphalt binder is passed through a sieve to separate particulate additives from the base asphalt. Particulate additives can result in potential separation or in interference with test sample geometry. In the current standard Superpave binder test methods, the particulate size is limited to 250 μm, selected arbitrarily as one-fourth (1/4) of the minimum testing sample dimension. The PAT separates material larger than 75μm using a No. 200 (0.075-mm) mesh. This size was selected because larger-size particulates are commonly considered part of the mineral aggregates in the asphalt mixture.

In the test, the asphalt binder is heated to 135°C until it becomes soft enough to pour. Approximately 10 ml of sample is transferred into a 125-ml Erlenmeyer flask. While hot, the sample is diluted using 100ml of solvent in small portions with continuous agitation until all lumps disappear and no undissolved sample adheres to the container. A metal, 50-mm diameter, No. 200 sieve disk is placed in the vacuum filtering apparatus, and the vacuum filtration is started. The container is washed with small amounts of solvent to facilitate filtering. Filtration is continued until the filtrate is substantially colorless. Then suction is applied to remove the remaining distillate. The material retained on the filtering sieve is transferred to a centrifuge tube, and the volume is measured partially filled with the solvent. The tube is placed in a centrifuge apparatus for 30 min at approximately 3,000 rotations per minute (rpm). At the end of the centrifugation, the volume of material at the bottom of the tube is measured to the nearest 0.01 ml. Using the final volume of particulates and the initial volume of sample, the percentage of compacted volume of the particulates retained on a No. 200 sieve by volume of the asphalt is calculated. The conditions used for the protocol were selected based on several experiments. The criteria used in interpretation of the PAT results are as follows:

- The test is conducted by using n-Octane as a solvent to determine the existence of an additive. If the test indicates that there is less than 2% by volume of material separated, the binder does not contain any additive. If the test indicates that there is more than 2%, the test should be repeated with toluene.
- The test is conducted by using toluene as a solvent. If the volume retained is more than 2%, the binder is not a simple binder due to existence of solid additives that are not likely to be soluble in asphalt.

<u>Test Results</u>

Table 2.1 gives results collected using the PAT for a selected grades modified with different additives. As can be seen, the highly modified binders such as B3 and B6 show significant amount of particular additives. These two binders have the high temperature of PG grade (PG 76-xx). In case of C6 (the binder of the same high PG grade), however, there is not much additive found after testing unlike B3 and B6. Although these two binders, B3 and B6, contain significant amount of solid additives beyond the maximum of the given criteria, the additional tests using Toluene show that these additives are soluble in Toluene and thus are dispersible in asphalt. None of the other binders show considerable particulate additives. The results shown indicate that the PAT test is an acceptable procedure that could be used to detect particulates. It also indicates that the binders used in Wisconsin do not contain high level of particulates. Those that contain a significant amount that is not soluble in octane, they are soluble in toluene and thus are mostly soluble or dispersible in asphalt. Since the effect of solid

additives needs to be considered, it is recommended that the PAT test is used as a screening test in future specifications for asphalts in Wisconsin.

Table 2.1 Summary of the PAT tests

Binder (Code)	PG Grade	Solvent	
		n-octane (%)	Toluene (%)
C5	PG 58-28	0.30	0.00
B3	***PG 76-28***	***39.0***	***0.00***
A3	PG 64-28	0.67	0.00
B2	PG 64-28	1.33	0.00
D1	PG 64-28	1.00	0.00
C4	PG 64-22	0.67	0.00
B5	PG 64-34	1.33	0.00
D2	PG 64-34	1.32	0.00
B8	PG 70-34	2.00	0.00
A1	PG 70-28	1.00	0.00
B4	PG 70-28	1.33	0.00
C2	PG 70-28	0.30	0.00
B6	***PG 76-34***	***43.0***	***0.00***
B7	PG 58-40	2.00	0.00
B9	PG 58-34	1.33	0.00
D4	PG 58-34	1.00	0.00
C6	PG 76-28	0.03	0.00
D5	PG 64-40	1.00	0.00
D6	PG 70-34	1.30	0.00

2.2 Laboratory Asphalt Stability Test (LAST)

The general requirements for a new test to evaluate the storage stability of modified asphalts were selected based on the review of research done in the past and on an evaluation of typical storage tanks and conditions used to store such asphalts in the field. If additives are detected in the binder based on the results of the PAT, it is required to evaluate the storage stability of these additives using the Laboratory Asphalt Stability Test (LAST). The values of maximum ratio of separation (Rs) and maximum ratio of degradation (Rd) are determined to describe the potential for storage instability. If any of the ratio is more than 120% or less than 80% at the critical separation time (T_{sc}) or critical degradation time (T_{dc}), the binder is not stable and is considered complex.

In this test two steps are required to evaluate the stability under two storage conditions: static and high agitation.

- Static Storage: Sample is kept at isothermal conditions with no thermal gradient and sampled at 0, 6, 12, and 24 hours.
- High Agitation Storage: Sample is kept at isothermal conditions, with high agitation speed, and sampled at 0, 6, 12, and 24 hours.

<u>Test Results</u>

Table 2.2 is an example of the result from running DSR tests after four times of periodic sampling for the binder C5. As can be seen, in case of this binder, there is not a critical potential of storage instability from storage tank without mechanical agitation. However, the sample experiencing agitation shows a potential of instability at the 6-hour sampling. Storage stability of this binder seems relatively stable compared to the other binder (B3) tested that is proved to be containing the particular additives from the PAT tests. The DSR tests reveal that B3 binder might have a problem of segregation at 6 and 24 hours' sampling under the high frequency of agitation testing condition.

In addition to the potential of segregation, sampling at the 6th hour indicates the potential of degradation under the high frequency (50 rad/s) condition. Therefore, those binders that have been sampled and tested need to be monitored during storage. More binders were tested and the data are included in Appendix I of this report.

This test method is rather long and requires extensive testing that is not suitable for quality control during production of hot mix asphalt. It is therefore recommended that testing is done by supplier at the time of modified asphalt production or during supply to the contractor. In addition, the testing under static (no agitation) conditions is similar to the Cigar tube test commonly conducted by suppliers. The LAST is therefore required only for agitation conditions to monitor possible break down of polymers. The TRB-FHWA binder task group has recommended that the test not be included in a specification due to its complexity and long time required.

Table 2.2 The Result of the LAST Tests for C5 Binder

Conditioning Step	External Heat without Agitation							
Frequency (rad/s)	5.0		50.0		5.0		50.0	
Temperature ©	HT		HT		IT		IT	
Time of Sampling (hr)	G*	delta	G*	delta	G*	delta	G*	delta
0 (top)	1,200	86.3	10,500	82.1	1,240,000	58.3	12,500,000	35.5
0 (bot)	1,300	85.9	11,200	81.6	1,240,000	57.2	11,200,000	34.9
6 (top)	1,360	86.1	11,800	81.8	1,320,000	57.0	13,300,000	34.3
6 (bot)	1,190	86.4	10,400	82.2	1,200,000	58.7	12,200,000	36.0
12 (top)	1,310	86.2	11,400	81.9	1,370,000	57.2	15,100,000	34.0
12 (bot)	1,190	86.4	10,400	82.2	1,210,000	58.8	12,500,000	35.7
24 (top)	1,160	86.4	10,100	82.2	1,080,000	59.8	10,200,000	37.4
24 (bot)	1,310	86.2	11,400	81.8	1,280,000	56.9	**11,500,000**	34.0
Time of Sampling (hr)	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d
0	0.92	1.00	0.94	1.01	1.00	1.02	1.12	1.02
6	1.14	1.00	1.13	1.00	1.10	0.97	1.09	0.95
12	1.10	1.00	1.10	1.00	1.13	0.97	1.21	0.95
24	0.89	1.00	0.89	1.00	0.84	1.05	0.89	1.10
Time of Sampling (hr)	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d
0	1.04	1.00	1.03	1.00	1.00	0.99	0.95	0.99
6	1.06	1.00	1.06	1.00	1.02	0.99	1.02	0.99
12	1.04	1.00	1.04	1.00	1.04	0.99	1.10	0.98
24	1.03	1.00	1.02	1.00	0.95	1.00	0.87	1.01

2.3 Workability of Binders

The need for reasonable temperatures for mixing and compaction is well recognized because safety and volatilization of binders are both impacted by high temperatures. The cost of energy for heating is another adverse impact of high temperatures. Viscosity is known to increase logarithmically with reduced temperature. The most commonly used method of plotting a viscosity-temperature profile is according to the ASTM standard method D2493. In this method the log-log of viscosity is plotted as a function of log temperature in degrees Kelvin (273+°C). For most asphalts, this relationship is linear and the Viscosity Temperature Susceptibility (VTS) index can be calculated as the slope of this relationship. The viscosity-temperature profile can, however, be considered truly valid only for asphalts whose viscosity is independent of the shear rate, a behavior described as Newtonian flow. For the majority of modified asphalts, and some high PG grade unmodified asphalts, viscosity is highly dependent on shear rate. It is believed that this shear dependency is what causes the difficulty of compaction of mixtures with modified binders.

In the current binder specification, there is no control on the binder's viscosity at the mixing and compaction temperatures typically used in the laboratory or in the field. There is a limit on viscosity at 135 C, which is considered as temperature for handling (pumping) but not for mixing and compaction. In the mixture design procedure the current guideline for using the high shear viscosity (6.8 1/s, 20 RPM) of 170 cP for mixing temperature and 280 cP for the compaction temperature. These requirements were evaluated in this project together with a new procedure of using low shear viscosity as discussed in the next section.

<u>Test Results</u>

The rotational viscometer was used to conduct the standard 20 rpm viscosity testing at three temperatures for each of the binders. The viscosity-temperature plots were used as described in the previous section to estimate the temperatures at which each binder achieves 170 cP and 280 cP.

Figure 2.1 shows the estimated compaction temperatures of all the tested binders using this guideline. Considering the reasonable limit of the compaction temperature of 150C, most of the binders are over this limit. Some of the binders show significantly unrealistic compaction temperatures which are over 200C.

In the NCHRP 9-10 project the concept of using the low shear viscosity, or zero shear viscosity (ZSV) was introduced due to the fact that most modified binders show much higher viscosities at lower shear rates. Low shear viscosity is found to be a better indicator of conditions during mixing and compaction and thus to insure proper coating and compaction, a maximum limit should be placed on the binder's low shear viscosity rather than the viscosity at 20 rpm.

The Zero Shear Viscosity (ZSV) is determined by using Brookfield Viscometer. The data from the viscometer are obtained by testing the binder at three temperatures: 105, 135, and 165°C, and at a series of different shear rates for each temperature. The testing is always done at increasing temperatures and shear rates. The shear rates range from 0.47 1/s to 93 1/s. The data are entered in an Excel spreadsheet and the solver program is used to arrive at the zero shear viscosity at each of the tested temperatures, using a fitting program. The $\log^2$ of the zero shear viscosities is plotted against the log of the temperature in degrees Kelvin.

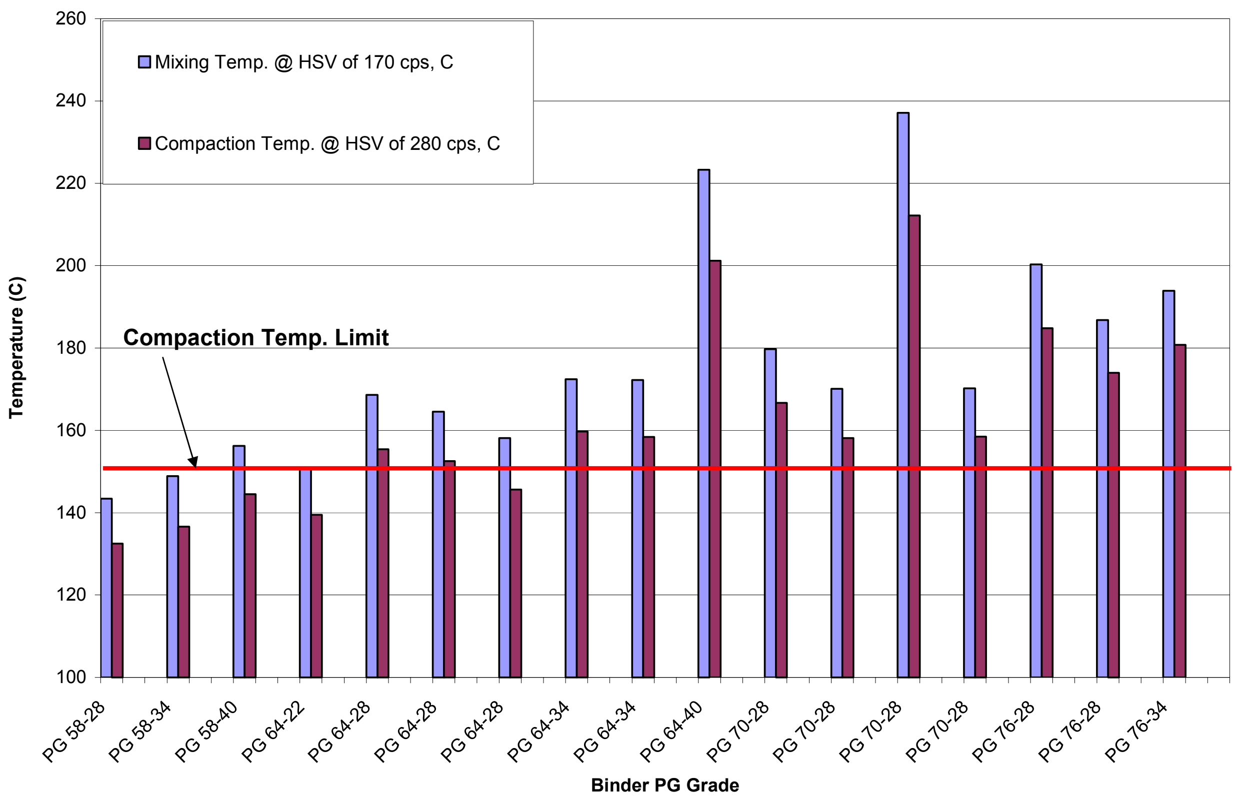


Figure 2.1 Estimated Compaction Temperatures Using a HSV of 280 cP

2.4 Development of Zero Shear Viscosity (ZSV) Guidelines

In the beginning of the study, the research team proposed the temperatures corresponding to a ZSV of 6.0 Pas and 3.0 Pas for compaction and mixing temperatures, respectively. However, this temperature selection guide created another disputable issue; after evaluating this criterion along with several other research institutes, it was questioned whether the estimated temperatures are applicable to the field mixing and compaction as shown in Figure 2.2. This Figure is the summary of all tested binders' mixing and compaction temperatures corresponding to a ZSV of 6.0 Pas and 3.0 Pas and most of the binders in this figure show relatively low compaction temperatures below the commonly used temperature of 150C. There is only one binder over this limit. Since the most modified binders require high mixing and compaction temperatures, the temperatures using this criterion seem to be underestimated.

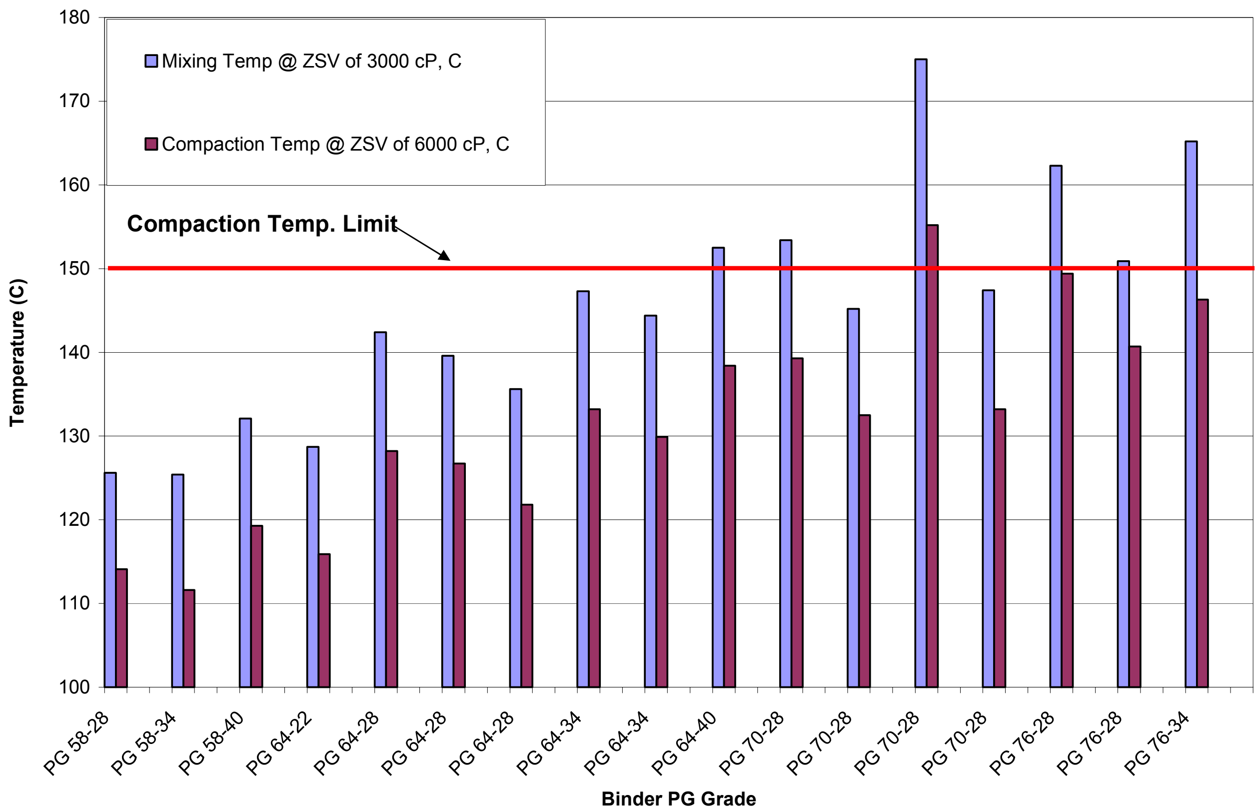


Figure 2.2 Estimated Compaction Temperatures Using a ZSV of 6.0 Pas

In addition, under this criterion of a ZSV of 3.0 Pas coating of aggregates with some binders has been reported to be difficult. In order to solve this problem, two methods could be suggested; one is to lower the limit of a ZSV from 6.0 Pas to 3.0 Pas and the other is to increase the asphalt content. Because increasing the asphalt content could be a concern, changing the limit of a ZSV has been evaluated. Figure 2.3 compares

HSV=280 cP, ZSV=6000 cP, and ZSV=3000 cP for the compaction temperature. This Figure indicates that using a ZSV of 3.0 Pas appears to result in the reasonable range for the compaction temperatures.

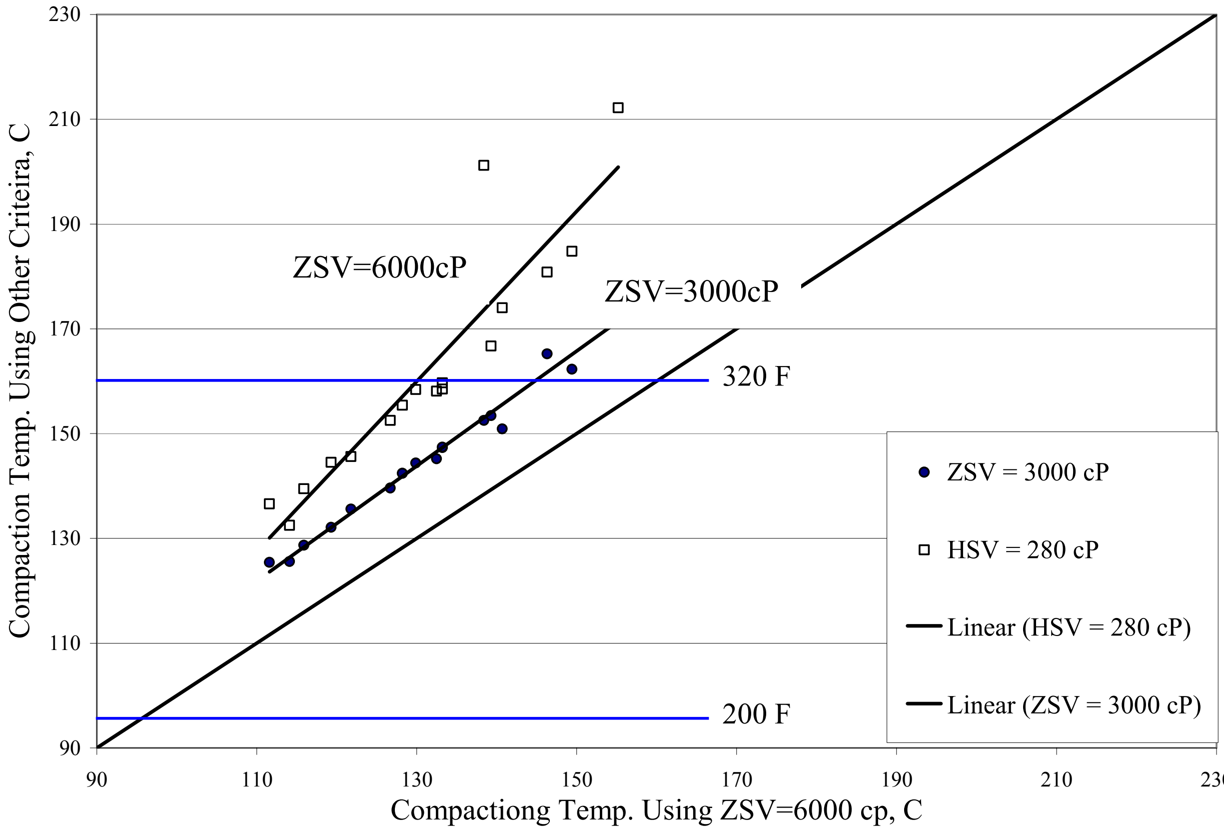


Figure 2.3 Comparison Between HSV=280 cP, ZSV=6000 cP, and ZSV=3000 cP

From the described research above, it can be concluded that using the ZSV of 1500 cP and 3000 cP for mixing and compaction temperatures, respectively, seems to allow more realistic mixing and compaction temperatures. The calculated ZSV with mixing and compaction temperatures, using the newly evaluated ZSV criterion, are given in Table 2.3.

From Table 2.3 it can be seen that the compaction temperatures obtained corresponding to 3.0 Pas within the rage of 125.4 – 175.0 C for the tested binders.

Table 2.3 Results of the ZSV Tests

Binder ID	PG Grade	Testing Temp C	η_0= (ZSV)	□□= (HSV)	Mix. Tº C ZSV 1500 cP	Comp. Tº C ZSV 3000 cP	Mix. Tº HSV 170 cP	Comp. Tº HSV 280 cP
C5	PG 58-28	165	455	75	138.5	125.6	143.4	132.5
		135	1803	243				
		105	10988	1345				
D4	PG 58-34	165	626	98	141.0	125.4	148.9	136.6
		135	1800	288				
		105	9002	1452				
B7	PG 58-40	165	700	121	146.5	132.1	156.2	144.5
		135	2501	443				
		105	14821	2601				
C4	PG 64-22	165	626	98	143.2	128.7	150.8	139.5
		135	2070	348				
		105	11988	2007				
A3	PG 64-28	165	1205	199	158.5	142.4	168.6	155.4
		135	3902	652				
		105	23963	4068				
B2	PG 64-28	165	925	167	154.1	139.6	164.5	152.5
		135	3936	650				
		105	23146	4408				
D1	PG 64-28	165	843	135	151.2	135.6	158.1	145.6
		135	3345	433				
		105	14996	2458				
B5	PG 64-34	165	1418	225	163.3	147.3	172.4	159.7
		135	5328	900				
		105	31752	5955				
D2	PG 64-34	165	1320	223	160.8	144.4	172.2	158.4
		135	4301	739				
		105	25727	4289				
D5	PG 64-40	165	1530	768	168.3	152.5	223.3	201.2
		135	9487	1985				
		105	39131	7769				
B8	PG 70-28	165	1799	297	169.3	153.4	179.7	166.7
		135	7500	1292				
		105	48407	8847				
B4	PG 70-28	165	208.9	1190	159.4	145.2	170.1	158.1
		135	876.8	4992				
		105	6400.1	38003				
A1	PG 70-28	165	4223	904	197.6	175	237.1	212.2
		135	13522	2345				
		105	11988	2007				
C2	PG 70-28	165	1477	223	163.5	147.4	170.2	158.5
		135	4954	810				
		105	33110	8150				
C6	PG 76-28	165	2623	582	176.9	162.3	200.3	184.8
		135	14390	2319				
		105						
B3	PG 76-28	165	1180	412	162.2	150.9	186.8	174.0
		135	11192	2057				
		105	126810	17852				
B6	PG 76-34	165	3150	565	186.7	165.2	193.9	180.8
		135	8641	2820				
		105	41303	28080				

Figure 2.4 summarizes all estimated mixing and compaction temperatures. It appears that most binders have the reasonable compaction temperatures that can be achieved easily in the field without the binders' experiencing thermal degradation. For some binders that have high compaction and mixing temperatures, relatively high temperatures both for mixing and compaction may be necessary in order to get enough workability before using the binder. It appears that A1, B6, and C6 are binders that could result in difficult mixing and compaction conditions. It is believed that these high temperatures are necessary because of their modification type that results in stiffer binders at high temperatures.

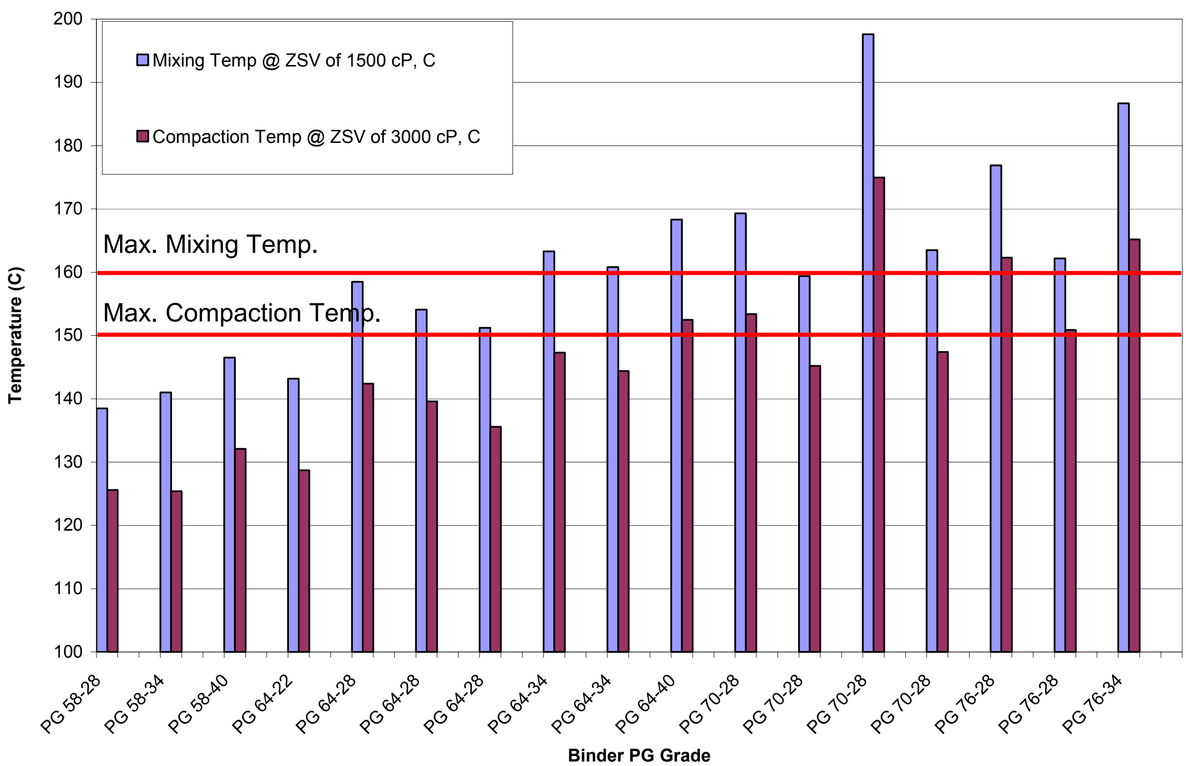


Figure 2.4 Mixing and Compaction Temperatures using ZSV of 3.0 and 1.5 Pas

Although intuitively, as high temperatures of the PG grade increase, both mixing and compaction temperatures increase as well, the data in Figure 2.4 show that this is not always the case. The data also show that within the same grade, a certain modifier can significantly affect such temperatures. For example, A1 whose grade is PG 70-28 has relatively high mixing and compaction temperatures even compared to the same PG grade binders such as B4 and C2. This binder (A1) shows a 20 - 30°C higher mixing and compaction temperature compared to the B4 and C2 binders.

2.5 Summary of Findings for Workability

Some of the important observations regarding the viscosity data are as follows:

- As expected, the non-modified binder, C5 (PG 58-28), shows relatively low mixing and compaction temperatures compared to all other modified binders except D4 (PG 58-34, SB modified).
- It appears that C5 (PG 58-28, non-modified) maintains the lowest zero shear viscosity at all temperatures while the A1 (PG 70-28, SBS modified) shows the highest viscosity. The B5 and D2 show a medium level of viscosity values.
- Using the ZSV target value of 6000 cps, the binders could be compacted at temperatures ranging between 111.6 C for D4 (PG 58-34, SB modified) and 155.1 C for A1 (PG 70-28, SBS modified).
- Within the same PG grades, there appears to be a significant difference in mixing and compaction temperatures. These differences are believed to be due to using different modifiers.
- Using the ZSV of 1500 cP and 3000 cP for mixing and compaction temperatures, respectively, seems to allow a more realistic estimation.

CHAPTER 3: BINDER RUTTING EVALUATION

3.1 Background

The findings of the NCHRP report 459 point out why the Superpave system fails to discriminate between the modification technologies that are successful and those which do not add value to the quality of binders. With respect to rutting resistance, the use of the total dissipated energy concept in deriving the $G^*/\sin\delta$, using cyclic reversible loading does not allow useful derivation of energy truly dissipated in permanent deformation. The repeated creep testing using the DSR was proposed in the NCHRP 9-10 project to measure binder response under cyclic non-reversible loading, which more accurately represents traffic loading conditions in the field. The test also allows separating energy dissipated in actual damage in terms of viscous - permanent flow from energy dissipated in delayed visco-elastic response, or what is called damping. The following sections include detailed analysis of the problems with applying the G*/sind to modified binders and the benefit of using the new repeated creep test. The chapter also includes the results collected for the binders of this study and a proposed grading system that could be used in a future specifications.

3.2 Problems with the Superpave Rutting Parameter

The main problems with the current Superpave specification parameter for rutting ($G^*/\sin\delta$) could be divided in 4 areas: (a) the kind of load used in testing (fully reversed), (b) the characteristics of the binder rutting parameter (based on total dissipated energy), (c) the number of cycles for testing (only few cycles of loading are used), and (d) the grade shifting for traffic speed and volume.

<u>Fully Reversed Load</u>

The fully reversed load applied currently in the DSR binder testing is not simulating in an appropriate way the mechanism in which rutting occurs in the pavement. The load that causes rutting in the real pavement is not fully reversed and is believed to be starting from zero rising to a maximum and returning to zero.

More important, fully reverse loading does not allow direct separation of energy dissipated in viscous flow and energy that is spent temporarily (rather than dissipated) in delayed elasticity (damping). The permanent deformation of the binder, which is the only contributor to rutting of asphalt mixtures, is represented by only the viscous component of the deformation when loaded. Figure 3.1 is prepared to clearly explain this important concept. In part (a), when the load applied is fully reversed, the permanent deformation is overcame and erased or included as part of the total strain making it very difficult to separate from delayed elasticity. In part (b), when one directional load is applied, the permanent strain can be easily separated from the elastic and delayed elastic strain.

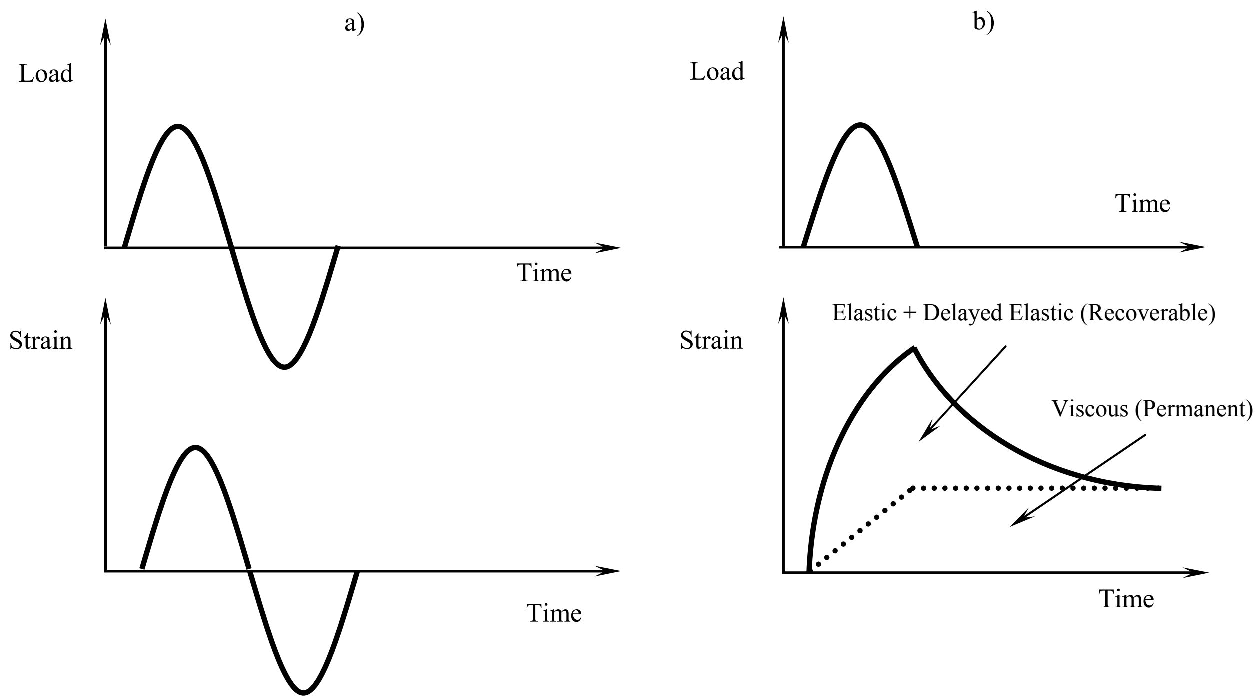


Figure 3.1 Permanent Deformation of the Binder under Unidirectional and Fully Reversed Loading

Total Energy Dissipated per Loading Cycle

The Superpave parameter is intended to control rutting by controlling the total energy dissipated per cycle (W_i). The formula for the calculation W_i in a constant stress testing is given by equation 1.

$$W_i = \pi \cdot \tau_0^2 \cdot \frac{\sin\delta}{G^*}; \qquad \text{eq.1}$$

where:

	W_i	= total energy dissipated in cycle i
	τ_0	= maximum stress applied
	G^*	= complex modulus
	δ	= phase angle

Higher values of $G^*/\sin\delta$ will result in lower Wi which is expected to result in less rutting. The relationship between W_i and the permanent deformation of the binder was validated under the SHRP research program for conventional binders. However, this relationship does not appear to be appropriate for modified binders. The problem with this concept is the fact that Wi can be divided into three components: elastic, delayed elastic and viscoelastic, as shown in equation 2.

$$Wi = W_{elastic} + W_{delayed\ elastic} + W_{viscous} \qquad \text{eq.2}$$

The part of the energy that contributes to permanent deformation is only the viscous, which is dissipated and not recoverable. The elastic and delayed elastic components are both recoverable so they do not contribute to permanent deformation. In conventional asphalts, the elastic and delayed elastic component is very small and thus the viscous component is very close to W_i at the high pavement temperatures. This is why W_i is a relatively good indicator of permanent deformation in conventional binders. However, modified asphalts are much more elastic or visco-elastic at high pavement temperatures and thus for modified asphalts W_i is not directly related to the energy dissipated in viscous flow, which results in permanent deformation. It is essential then to isolate the viscous component of W_i in order for accurate estimation of the rutting resistance of a modified binder.

To demonstrate the concept, Figure 3.2 shows the result of a creep test on three binders. One of them is a conventional binder (oxidized) and the other two are binders modified with polymers. The Figure shows clearly how the conventional binder has a very low recovery during the unloading cycle. On the other hand, the modified binders present a bigger elastic component, with a magnitude greater or similar to the viscous component for both of the asphalts.

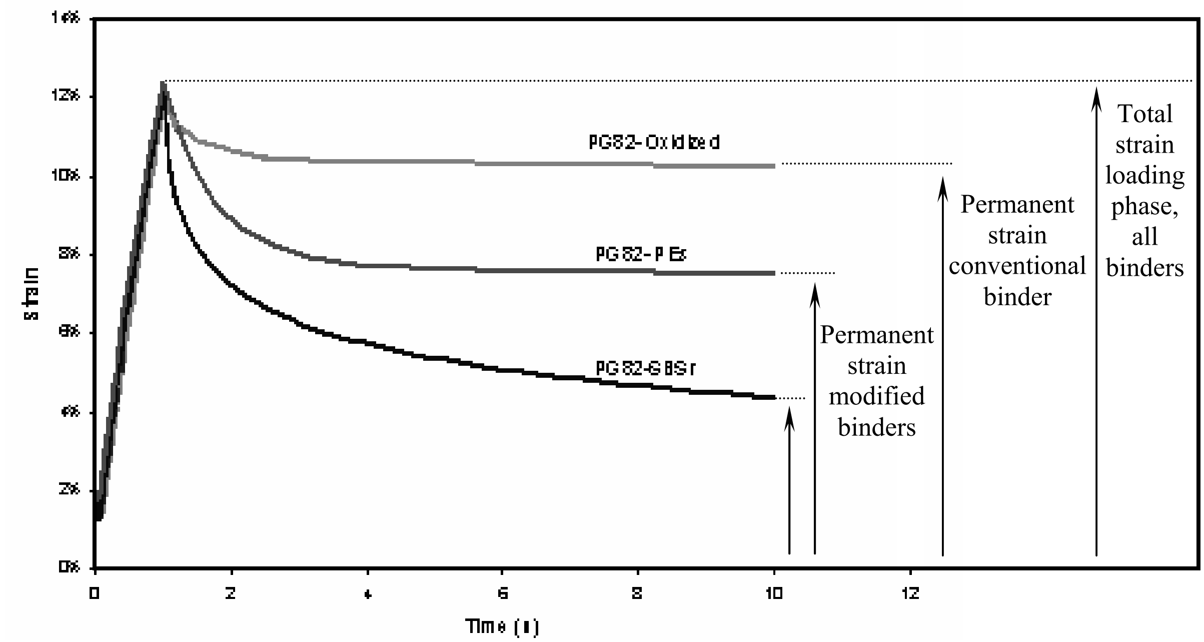


Figure 3.2 Recovering Capacity of Conventional and Modified Binders under Creep Testing

<u>Number of Loading Cycles</u>

The G*/sinδ is calculated after testing for only a few loading cycles. The damage accumulation (permanent deformation) occurs after many cycles of loading. Asphalts that have similar performance after a few cycles of loading can have very different accumulated deformation after thousands of cycles because of the memory effects and the vico-elastic nature of asphalts, particularly modified asphalts. The number of cycles needed for an accurate characterization of the binder resistance to rutting must be enough to reach a steady state behavior. This would allow extrapolating the results to the number of cycles required. In the new repeated creep test a minimum of 50 cycles were found to be necessary to reach, with a certain security, a nearly steady state.

<u>Traffic Speed and Volume</u>

In the Superpave specification low speed and high volume traffic are taken into account simply by shifting the grade of the binder in the high pavement temperature. Since it has been proved that modified binders of the same grade can have very different sensitivity to loading time and temperature, the grade bumping is not an appropriate way of taking into account the time of loading (traffic speed and volume). Also, testing at a different temperature (for example specifying a PG 64 for a high pavement temperature of 58°C) does not represent the actual behavior at pavement design temperature. There is therefore a need for determining a specification parameter that considers in a more rational way the effect of total loading time in permanent deformation.

3.3 A New Binder Parameter for Rutting

During the NCHRP 9-10 research program a new new parameter was selected for characterizing the ability to resist rutting of binders, called viscous component of the creep stiffness G_v. This parameter is measured testing the binder under repeated creep constant load. The test can be easily carried out programming the DSR in the repeated creep mode. With this new parameter, the problems presented by $G^*/\sin\delta$ are overcome.

Cyclic Creep Loading

In the new repeated creep test a cyclic load in one direction is applied. Figure 3.3 shows the loading pattern applied and the typical behavior of a modified asphalt for three cycles of loading. The creep test allows isolating the viscous (permanent) deformation from the elastic and delayed elastic (recoverable). In this way the ability to resist permanent deformation of the asphalt can be truly characterized.

Number of Loading Cycles

The typical behavior of a binder under cyclic creep testing shows a first stage were the permanent deformation per cycle varies from one cycle to the other reflecting the effect of delayed elasticity. However, after around 30 cycles of loading, the rate of deformation per cycle tends to stabilize reaching a steady state. It is important to reach this steady state to accurately extrapolate the susceptibility to long term damage of the binder.

An example of such results is shown in Figure 3.4 During the first cycles of loading, the asphalt PG 64-28 SBS modified shows a higher rate of permanent deformation than the PG 64-28 Elvaloy modified. However, when the steady state is reached, the PG 64-28 SBS modified shows less rate of permanent deformation than the

PG 64-28 Elvaloy modified. This indicates that the PG 64-28 SBS modified has better rutting resistance than the PG 68-28 Elvaloy modified. Also it is seen that both binders accumulate much less permanent strain compared to the third binder which is SB modified. It is important to notice that all three binders are of the same PG grade (64-28), which signifies the failure of the current system to capture the differences between their rutting performance.

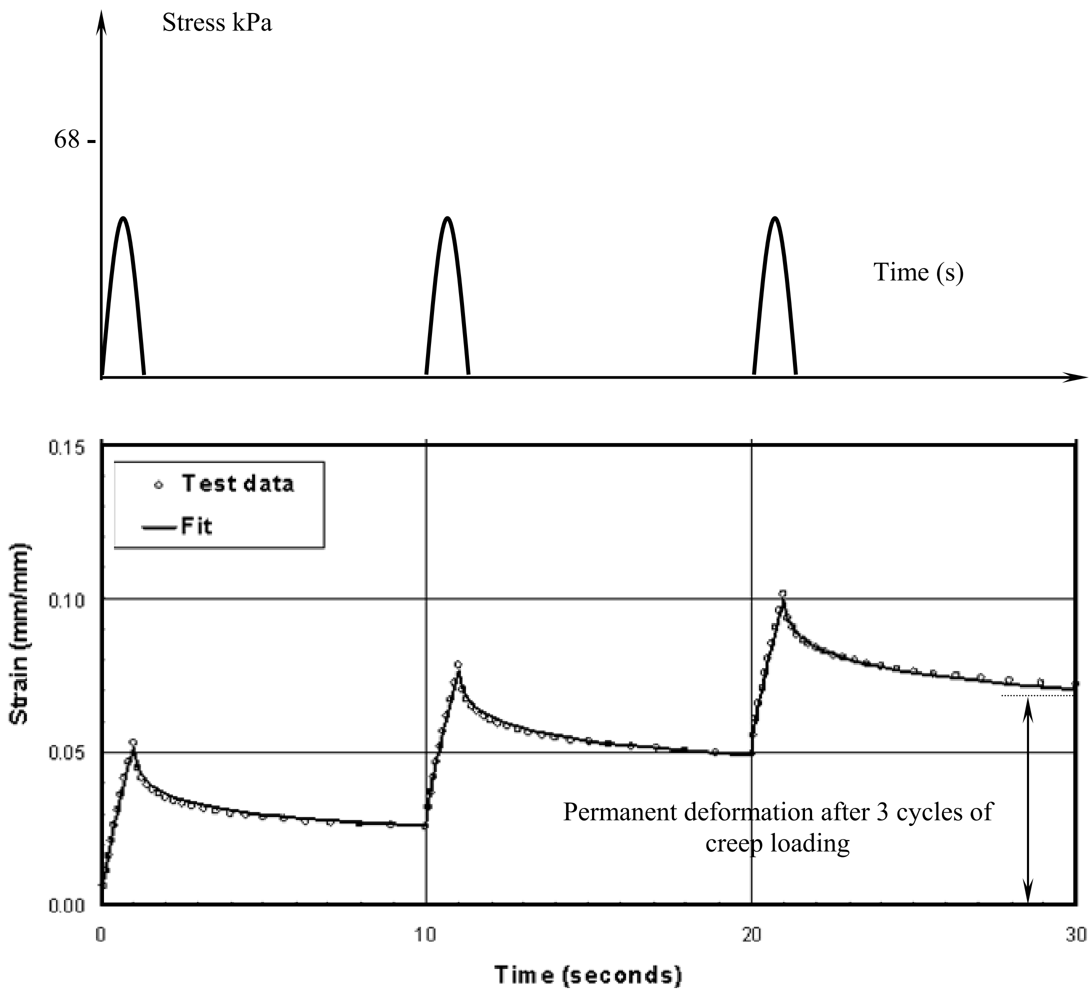


Figure 3.3 Loading Pattern and Binder Strain under Repeated Creep Test

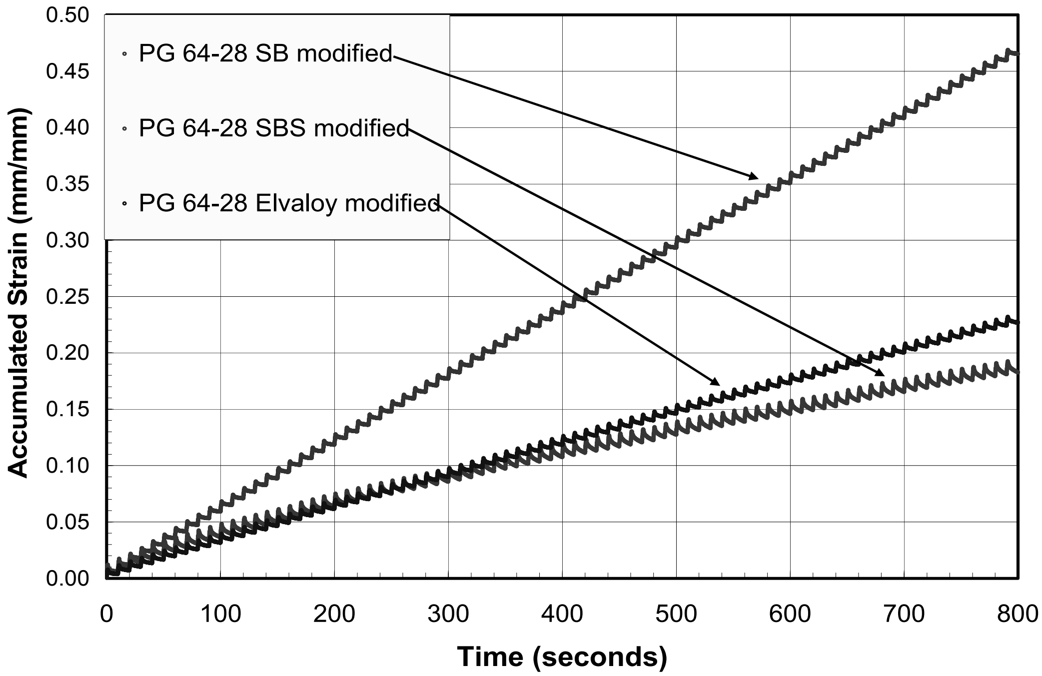


Figure 3.4 Behavior of Binders of the same PG grade Under Repeated Creep Loading

Determination of the Viscous Component of the Creep Stiffness G_v

When the steady state is reached, then G_v is calculated. The behavior of the asphalt is modeled using the four element Burger model. G_v is determined by fitting the Burger Model to the data set from cycles 50 and 51, Based on the results of the testing of a large number of binders, it was proved that cycles 50 and 51 give a reasonable security that the steady state is already reached.

The total shear strain versus time is expressed by equation 3.

$$\gamma(t) = \gamma_1 + \gamma_2 + \gamma_3 = \frac{\tau_0}{G_1} + \frac{\tau_0}{G_2}\left(1 - e^{-t/\tau}\right) + \frac{\tau_0}{\eta_3} t \; ; \qquad \text{eq.3}$$

where:

$\gamma(t)$	= total shear strain
$\gamma 1$	= elastic shear strain
γ_2	= delayed elastic shear strain
τ_0	= constant stress
G_1	= elastic component of the complex modulus

G_2 = delayed elastic component of the complex modulus

η_3 = non recoverable steady state viscosity

G_v is defined as the viscosity η_3 divided by the loading time (t), as shown by equation 4.

$$Gv = \eta_3 / t \qquad \text{eq.4}$$

Traffic Speed and Volume

It was determined during the NCHRP 9-10 research that, considering the accuracy of the rheometers currently used, the most appropriate loading time for the creep testing was 1 second. Even when the loading times that would correspond to the real traffic speeds are shorter (0.1 to 0.01 seconds), most of the rheometers are not able to achieving a true creep test with such short loading times. Thus the value of G_v measured by the cyclic creep testing correspond to the G_v at 1 second. This value has to be transformed to the value corresponding to the actual traffic speed. This conversion is carried out by simply introducing the corresponding time of loading in equation 4. For example, if the G_v value measured at 1 second loading time is equal to A, the corresponding Gv value for 0.01 seconds would be equal to A divided by 0.01.

The traffic speed accounted by these means represents an important improvement from the grade bumping currently used in the PG grading system. The multipliers used for G_v are obtained assuming that the permanent deformation due to one loading cycle of 1 second is equal to the permanent deformation produced by 10 loading cycles of 0.1 second. This is a more rational and scientific method for taking into account longer loading times resulting from slower moving traffic.

To account for traffic volume a similar approach is used. The addition of more loading cycles will results in more total loading time. In other words 100 trucks moving at 60 mph will result in 10 times the loading time of 10 trucks moving at the same speed. The limits for the G_v can therefore be simply calculated for a specific traffic volume and changed proportionally to the other traffic volumes. This will be explained in a later section with the implementation to Wisconsin conditions

3.4 Testing to Derive Limits for G_V

Test Information

A total of 19 binders were tested during the present project, which varied in PG grade (from PG 58 to PG 76) and modifier type used (SBS, SB, and Elvaloy modified). Both original (non-aged) and RTFO (short term aging) aged binders were testes. The value of G_v was determined for each of the binders. Important information about the testing is listed below:

- Testing temperature = 58°C (high pavement design temperature in Wisconsin, 98 % confidence level).
- The DSR sample was prepared according to the AASHTP TP5 standard procedure. The stress stress-controlled rheometer is programmed to run a repeated creep test of a total of 100 cycles of 1second loading and 9 seconds unloading.
- Most modified binders are stress sensitive and thus a minimum stress level was required. Micro mechanics analysis of images of typical mixtures has shown that stress level in binder under a typical 600 kPa truck tire is not high and could be in the range of 10 to 50 Pa. To avoid confounding stress non-linearity, a starting stress of 25 Pa was used.
- Since model fitting is iterative and can converge to give several combinations of the parameters, it is important that the fitted and measured total accumulated strains at 100 cycles are compared. If the difference is more than 5 %, the fitting procedure should be continued for more iterations and better match of the total accumulated strain at 100 cycles.

Summary of Repeated Creep Results

The values measured for G_v at the selected pavement design temperature are shown in Table 3.1. Most of the binder showed an increment in G_v after the RTFO aging. However, five of the binders (C6, A1, D5, B6 and C2) showed a lower value of the G_v for the RTFO aged residue compared with the original binder. This is not surprising, since in the literature has been pointed out that the RTFO aging can provoke degradation in some polymers used in binder modification. However these materials can not be considered as well performing materials so they should not be used for setting specifications up. On the other hand, it is not completely clear that these materials would soften in the real mixing and construction process. For these reasons, the RTFO aged G_v values for the binders that softened after the RTFO aging were excluded from the analysis carried out in the following sections.

Table 3.1 Measured G_v Values at 58°C and 1.0 Second Loading Time

Binder ID	PG Grade	Modification Type	Measured Gv (kPa)	
			Original	RTFO aged
C5	PG 58-28	SBS	13	33
B9	PG 58-34	Elvaloy	73	183
D4	PG 58-34	SB	19	50
B7	PG 58-40	Elvaloy	39	126
AVERAGE PG 58			**36**	**98**
C4	PG 64-22	SBS	62	101
D1	PG 64-28	SB	111	281
B2	PG 64-28	Elvaloy	88	444
A3	PG 64-28	SBS	2380	2550
D2	PG 64-34	SB	345	659
B5	PG 64-34	Elvaloy	232	918
D5	PG 64-40	SB	1190	658*
AVERAGE PG 64			**630**	**826**
A1	PG 70-28	SBS	3670	1450*
C2	PG 70-28	SBS	425	137*
B8	PG 70-28	Elvaloy	211	607
B4	PG 70-28	Elvaloy	131	591
D6	PG 70-34	SB	2290	3960
AVERAGE PG 70			**1345**	**1719**
B3	PG 76-28	Elvaloy	1220	3880
C6	PG 76-28	SBS	8870	5540*
B6	PG 76-34	Elvaloy	652	372*
AVERAGE PG 76			**3581**	**3880**

* Values excluded from the analysis

Without considering the binders that softened after primary aging, A3 (PG64-28 SBS modified) and D6 (PG70-34 SB modified) appear to perform the best among the tested binders, in terms of G_v. It is highly remarkable that the binders that were expected to perform the best in terms of rutting (PG76-xx) were not necessarily the most outstanding in terms of performance. Comparing binders of the same high temperature PG grading, it appears that the values of G_v vary highly depending on the modification type. For instance, the binders of PG 64-xx have significant variations in the original binder G_v values ranging from 62 kPa to 2380 kPa. This is more than 38 folds of the lowest value estimated for the same grade. The difference is similar for the RTFO aged Gv values of the PG64-xx binders. For the PG 70-xx binders, the variation in original binder G_v ranges form 131 kPa to 2290 kPa, which is a 17 fold difference. It is no wonder that there are significant concerns about the PG grading system.

Despite the big differences between the G_v values of binders of the same high temperature PG grade, a trend could be identified. As the high temperature of the PG grade increases, the average value of G_v increases as well. This trend was used to derive the field conversions factors, as will be discussed in a later section.

3.5 Derivation of Specification Criteria

Mechanistic Binder Specification Framework

In order to use the binder rutting results in a specification framework, the application conditions should be clearly defined. For rutting resistance, the framework should be based on high pavement design temperature, traffic speed and traffic volume, which is similar to the concept in the current Superpave practice . The effect of traffic conditions (speed and volume) are, however, considered by changing the limits of the Gv value rather than by grade shifting on the temperature scale. Table 3.2 depicts such a framework in which traffic speed and traffic volume are explicitly included in the grading system. This could be called a mechanistic approach for selecting asphalts since the critical conditions that affect the mechanics of permanent deformation are considered in selecting the grade of the asphalt. In the following sections an example of how such a system can be implemented in Wisconsin is presented.

Weather and Traffic Information

According to the SHRP Superpave weather database most areas of Wisconsin belong to the PG 58-xx pavement temperature region. For this reason, the research team decided to stay with one high pavement temperature: 58°C (98% confidence level). This way, complexity of selecting binders is expected to be reduced.

Six levels of traffic volume are recognized on Wisconsin highways and are used in the current asphalt mixture design activities. The traffic levels considered are shown in the first column of Table 3.3, in accordance to the PG Binder Selection Criteria. Two levels of traffic speed were selected. High speed, assumed to be 60 mph (0.01 seconds loading time), which is 5 miles per hour less than the maximum allowable traffic speed in Wisconsin. Low speed, assumed to be 15 mph (0.04 seconds loading time), for taking into account the slow movements of traffic in urban areas.

Table 3.2 Example of a Mechanistic Binder Specification Framework for Rutting Resistance that Includes Pavement Design Temperature and Traffic Conditions without Grade Shifting

Purpose	Test Parameter	AASHTO Method	Testing Rate	Testing Stress	Testing Temp.° C	Criteria	Traffic Level (Millions ESALs)		
							L	M	H
							<1.0	1.0-3.0	>3.0
Test on Original Binder									
Rutting Resistance	Gv	TP5	Load/ unload	Creep Stress			G_v at loading time		
Traffic Speed: F			0.01/ 0.09 s	25 Pa	HT	Minimum	(c)	(d)	(e)
Traffic Speed: S			0.1/ 0.9 s	25 Pa	HT	Minimum	2x(c)	2x(d)	2x(e)
Test on RTFO Aged Binder									
Rutting Resistance	Gv	TP5	Load/ unload	Creep Stress			G_v at loading time		
Traffic Speed: F			0.01/ 0.09 s	25 Pa	HT	Minimum	2x(c)	2x(d)	2x(e)
Traffic Speed: S			0.1/ 0.9 s	25 Pa	HT	Minimum	4x(c)	4x(d)	4x(e)

Note: In the best judgement of NCHRP9-10, the following limits are recommended:
(c) = 20 kPa, (d) = 60 kPa, (e) = 200 kPa

Table 3.3 Traffic Levels and Applicable PG Grades for High Traffic Speed

ESALs	Applicable Binder High Speed	Average ESALs
0 - 0.3	PG 58	500,000
0.3 - 1	PG 58	
1 - 3	PG 64	5,000,000
3 - 10	PG 64	
10 - 30	PG 70	20,000,000
30 -	PG 76	50,000,000

Deriving Field Conversion Factors

A necessary condition for defining new specifications is taking into account the past experience in the field. The new parameters must be founded on reliable data coming from the performance of the binders in the field, and not only from theoretical derivations. That's why, despite of the fact that the Superpave system has shown critical gaps, it was assumed that using the PG grading experience in Wisconsin is the most suitable and realistic way of relating the binders to the field performance. Wisconsin has been using PG graded binders for at least 7 years with relatively good success evident by lack of rutting problems. It is therefore reasonable to assume that the relationship between the PG grade and the level of traffic for which it is used is a good starting point for determining which traffic level these binders are suitable for. Table 3.3 shows the applicable binder for each of the traffic levels at standard (high) traffic speed according to the PG Binder Selection Criteria developed by Wisconsin DOT. The third column of Table 3.3 shows the average traffic level for each of the PG graded binders. These average values were considered the hypothetical allowable ESALs for each of the PG grades at fast moving traffic.

Using the data collected in this project, the G_v values for each set of the high temperature PG grades were averaged. The average G_v values measured at 1 second loading time are shown in Table 3.1. The G_v values corresponding to high traffic speed can be calculated for each of these averages by dividing them by a factor of 0.01. The relationship between the average allowable ESALs of each PG grade and the average G_v value at high speed for each PG grade defines the field conversion factors used for constructing the specification criteria. Figure 3.5 shows the relationship between average G_v value fro each PG garde and the allowable ESALs for the original and RTFO aged binders (high traffic speed). The field conversion factors can be represented by the slope of the trend lines shown in the Figure 3.5. It is important to note that the RTFO aged binders show a slightly different trend line compared to the unaged. It is recommended that this difference is maintained in the specification because modified binders vary in their reaction to short term aging.

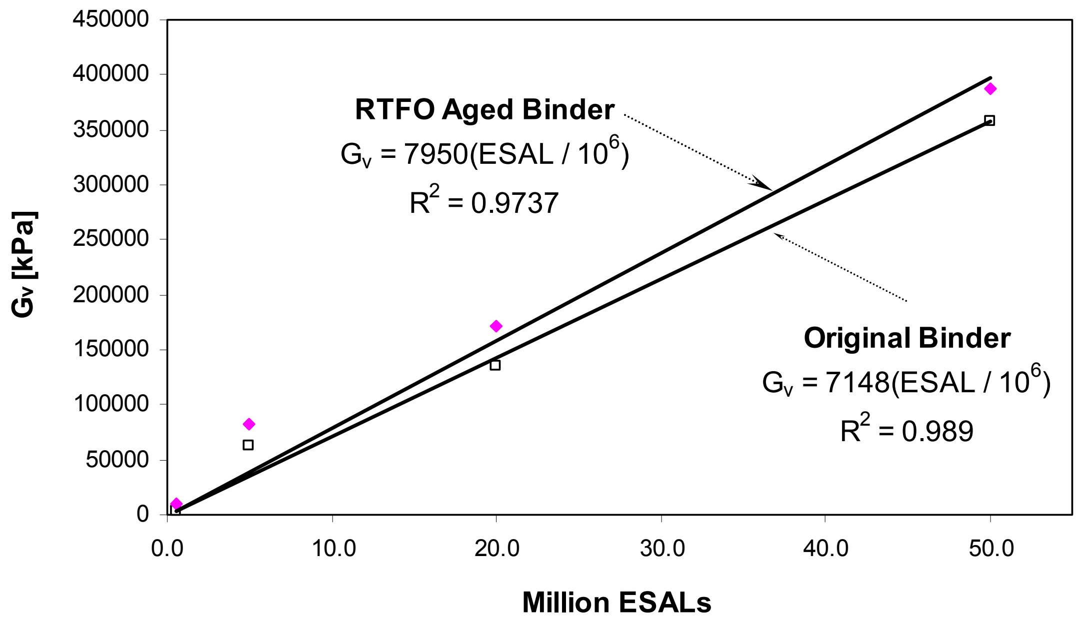


Figure 3.5 G_v versus Allowable ESALs, Original Binder, High Traffic Speed (0.01 s loading time)

3.6 Proposed Binder Criteria for Rutting

Using the relationships shown in Figure 3.5, the proposed binder criterion for rutting was derived by specifying a minimum G_v value for each of the traffic categories defined in the PG Binder selection criteria. Criteria are defined for high and slow speed. The G_v values for high speed are obtained by multiplying the average volume of each traffic range by the corresponding field conversion factor (original of RTFO). For example, the minimum value of G_v specified for the traffic range from 1 to 3 millions ESALs is the one calculated using 2 million ESALs. The G_v values for low speed are obtained from the G_v values for high speed. The speed conversion factor in this case is 4, which represents the ratio between the loading time for fast traffic (0.01 s) and the loading time for slow traffic (0.04 s). Table 3.4 shows the calculated minimum values for each traffic volume and traffic speed. A binder should meet both G_v values, original and RTFO aged, at the corresponding speed for being suitable for a specific traffic volume.

Table 3.4 was used for constructing the proposed specification for rutting. The calculated Gv values were rounded before being included in the specification. Table 3.5 shows the proposed specification for the Wisconsin state, which follows the framework presented before in Table 3.2. It provides a farmework for selecting a binder suitable for a specific traffic speed, traffic volume and high pavement temperature, which is a mechanistic system.

Table 3.4 Proposed Binder Rutting Criteria and Ranking of Binders

Speed	Volume Range (million ESAL)	Average Volume (million ESAL)	Minimum Gv [kPa] Original Binder	Minimum Gv [kPa] RTFO Aged Binder	Applicable Binders
Fast *	0 – 0.3	0.15	1072	1193	C5(PG58), D4(PG58), B7(58)
	0.3 – 1	0.65	4646	5168	B9(58), C4(64), D1(64), B2(64), B4(70)
	1 – 3	2	14296	15900	D2(64), B5(64), B8(70)
	3 – 10	6.5	46462	51675	B3(76)
	10 – 30	20	142960	159000	
	> 30	30	214440	238500	A3(64), D6(70)
Slow **	0 – 0.3	0.15	4289	4770	D2(64), B5(64), B8(70)
	0.3 – 1	0.65	18585	20670	B3(PG76)
	1 – 3	2	57184	63600	A3(64), D6(70)
	3 – 10	6.5	185848	206700	
	10 – 30	20	571840	636000	
	> 30	30	857760	954000	

Notes: * Gv for fast traffic is obtained dividing the measured value by 0.01
* Gv for slow traffic is obtained dividing the measured value by 0.04

Table 3.5 Tentative Specifications for Rutting

Purpose	Testing Rate	Testing Stress	Testing Temperature	Traffic Level (Millions ESALs) 0 - 0.3	0.3 - 1	1 - 3	3 -10	10 - 30	> 30
Test on Original Binder									
Rutting Resistance	Load / unload	Creep Stress		Minimum G_v at loading time (MPa)					
Traffic Speed: F*	0.01/ 0.09 s	25 Pa	58°C	1.0	5.0	15.0	50.0	150.0	> 200.0
Traffic Speed: S**	0.04/ 0.36 s	25 Pa	58°C	4.0	20.0	60.0	200.0	600.0	> 800.0
Test on RTFO Aged Binder									
Rutting Resistance	Load / unload	Creep Stress		Minimum G_v at loading time (MPa)					
Traffic Speed: F*	0.01/ 0.09 s	25 Pa	58°C	1.25	6.0	18.0	60.0	180.0	> 250.0
Traffic Speed: S**	0.04/ 0.36 s	25 Pa	58°C	5.0	24.0	72.0	240.0	720.0	> 1000.0

3.7 Ranking of Binders

Using the criteria proposed the binders tested could be ranked according to their contribution to rutting resistance. Table 3.4 shows how the tested binders fit into each of the traffic levels and traffic speeds. It can be seen that the ranking of the binders with respect to the G_v value has no good correlation with the PG grading. For fast speed and traffic level between 0.3 – 1.0 million ESALs binder from three different PG grades are found (PG64, PG70 and PG76). The PG grades suitable for the higher traffic level and high speed are not PG76-xx, as it was expected; they are PG64-xx and PG70-xx. This is a clear confirmation that the grade shifting due to traffic speed or traffic volume is not a correct tool to take into account the time of loading, i.e. traffic conditions.

Only 6 binders showed adequate rutting resistance for slow traffic conditions. None of the binders studied would be appropriate for being used in traffic levels higher that 3 million ESALs at slow speeds. This is explained by the fact that the low speed considered (15 mph) represents an increase in the time of loading of 4 times for each load repetition. For this reason the G_v value required for slow traffic is 4 times higher than that required for fast traffic, for the same traffic volume. In the current PG grade shifting it is assumed that one grade shift is needed to go from high speed to slow speed, which is less than the factor of 4. The appropriate binders for low speed determined using this mechanistic approach are expected to be different from the ones that would have been obtained using the simple PG grade shifting.

3.8 Summary of Findings for Rutting

- Using the total dissipated energy to rank binder according to their resistance to permanent deformation could be misleading, particularly for modified asphalts. For conventional asphalts the total energy dissipated measured by $G^*/\sin\delta$ and the energy dissipated in viscous flow could be similar and thus could be a good approximation. For binders with high visco-elasticity, the difference between total energy and energy dissipated in viscous flow could be very large and thus $G^*/\sin\delta$ cannot rank binder correctly with respect to rutting resistance.

- To estimate energy dissipated in viscous flow, a non-reversible cyclic loading is needed. Such loading is attained by the repeated creep testing in which an asphalt is allowed to recover and thus physically separate permanently dissipated energy from delayed elastic stored energy.
- Comparing binders of the same PG grades, it appears that the values of G_v vary highly depending on the modification type. Because of the significant range in G_v for these binders, the concerns about the PG grading system not being able to identify better performing modified asphalts are valid.
- Modified binders can deteriorate after RTFO aging. The proposed guidelines suggest the checking of the G_v value before and after primary aging. This procedure allow rejecting materials that soften too much after aging. Although the consequences of this softening are not known in terms of pavement performance, it could mean degrading of polymers, which cannot be considered beneficial.
- The criteria proposed is based on field conversion factors derived from a limited amount of data. The limits are derived based on assumptions and approximations that may or may not be completely valid. The limits are considered as a starting point for further evaluation by the State Highway Agencies and the Industry. It is hoped that a major national study will be initiated to collect enough data to derive more reliable field conversion factors.
- The criteria proposed allow consideration of traffic speed and volume without grade bumping. The adjustment factors for these conditions are derived based on field conversion factors and mechanistic understanding of the parameters used in the criteria.
- The proposed grading system is expected to allow the agency and the supplier to determine what the traffic conditions are that this asphalt is suitable for without artificially changing temperatures of testing. A PG 58 –28 could be of such a high quality that it would fit high and low traffic volume and fast and slow speed without the need to harden it and use a PG 64-28 grade. If there is a need for modification, then that modification will be specifically designed to solve a non-acceptable behavior such as sensitivity to traffic speed and traffic volume.

CHAPTER 4: FATIGUE EVALUATION OF BINDERS

Similar to the previous chapter, the problems with the current fatigue parameter will be first presented followed by testing results and proposed criteria for controlling fatigue of binders and selection of grades.

4.1 Problems with Superpave FATIGUE Parameter

The current Superpave specification parameter for fatigue (G*sin δ) has some problems that can be grouped into the following areas: (a) it is good only for strain controlled conditions; (b) is based on the energy dissipated per loading cycle; (c) it is obtained after only a few cycles of loading and (d) it does not consider the type of pavement structure.

Strain Controlled Conditions

The Superpave parameter G*sinδ is derived from the total energy (Wi) dissipated per loading cycle under a strain controlled loading. Equation 1 shows the relationship for Wi under strain controlled conditions.

$$W_i = \pi \cdot \gamma_0^2 \cdot G^* \cdot sin\delta \, ; \qquad \text{eq.1}$$

where: W_i = total energy dissipated in cycle i

γ_0 = constant strain applied

G^* = complex modulus

δ = phase angle

Fatigue is supposed to be minimized by controlling W_i. Lower values of G*sinδ would result in lower energy dissipated per loading cycle under strain controlled conditions. Fatigue is known to be more critical in weak pavements, where the loading is believed to be strain controlled. However, the limit between weak pavement and strong pavement is not very clear. I would be much more useful to have a parameter that would be independent of the mode of loading because it is very difficult to decide what will be the conditions of the pavement during selecting of the binders. A parameter applicable for all kind of pavement structures would provide a more reliable and simple prediction for the fatigue life of roads.

Dissipated Energy per Loading Cycle (Wi)

The energy dissipated per loading cycle Wi does not necessarily represents the tendency of the binder to having a poor fatigue performance. Under cyclic, reversible loading the energy (Wi) can be dissipated into two components: visco-elastic damping and irreversible damage. Thus, Wi represents energy that is used by the time dependent behavior of the material or energy that is dissipated in either viscous flow or in cracking. During cyclic reversible loading it is very difficult to separate these mechanisms and thus the simple measure of Wi for a few cycles is not a measure of cracking damage that could occur due to fatigue. This was proven experimentally by considering the correlation between $G^*\sin\delta$ and the fatigue life of mixtures, which was proven to be very low. The solution to this challenge is to consider the change in Wi values because the relative change due to repeated cycling is a good physical indicator of the initiation of the fatigue damage and the progression.

Numbers of Loading Cycles

The value of $G^*\sin\delta$ is derived from Wi calculated after testing for only a few loading cycles. The damage accumulation (fatigue) happens after many cycles of loading. This fatigue behavior can be shown for example like a decrease in the G* value of the binder. Two asphalts can have very similar performance after a few cycles of loading but they can defer highly when thousands of cycles are applied. Figure 4.1 shows the G* values for some binders as a function of the cycles of loading for strain controlled conditions. It can be see that all the binders show a steady behavior in terms of G* values for the first thousand cycles. However there is a certain point when G* starts to decrease. This is a sign of damage occurring in the binder. It is also shown that binders that had the same G* value at the beginning of the testing, start to deteriorate at different number of loading cycles. This means that they have different fatigue lives.

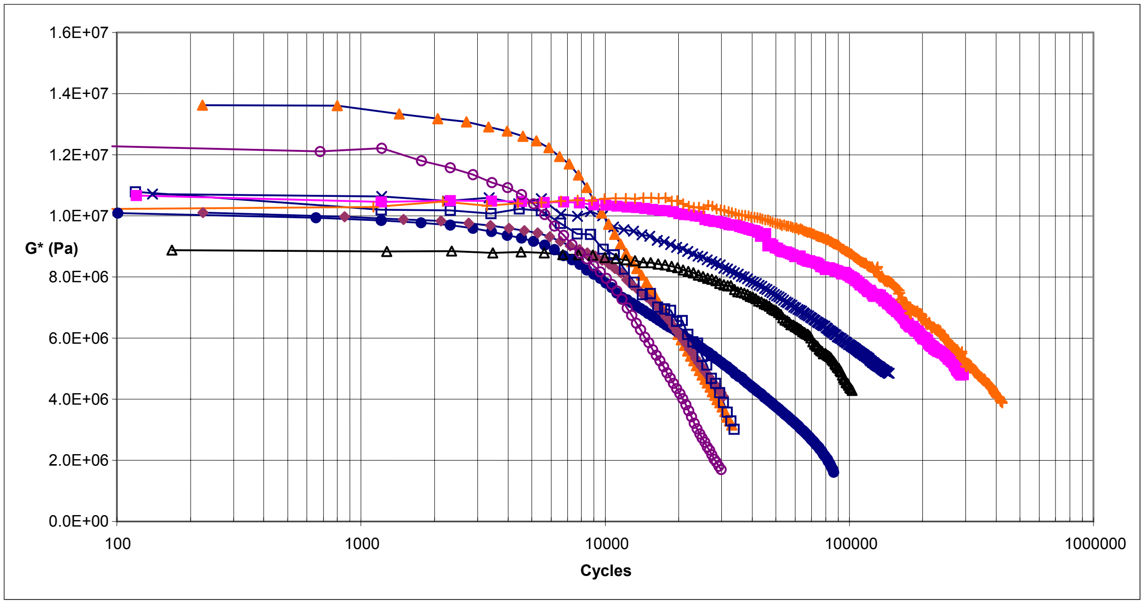


Figure 4.1 G* vs. Loading Cycles for Strain Controlled Testing

<u>Pavement Structure</u>

Fatigue behavior in asphalts is highly dependent on the level of load applied to the binder. Higher strains (or stresses) lead to earlier fatigue damage than lower ones. That is why fatigue damage is usually more common in weak pavements, where the strain level is higher. The current parameter G*sinδ does not consider the load applied to the binder in the characterization of the fatigue behavior. This way, the differences between weak structures and strong structures are neglected in the current specifications, which is a high inaccuracy.

4.2 A New Binder Parameter for Fatigue

A new fatigue parameter was proposed in the NCHRP 9-10 research program for determining the fatigue resistance of binders. The new parameter is obtained after testing the asphalt in the DSR binder for a large number of cycles until damage is shown in the sample. The type of loading is the same as applied in the current Superpave fatigue testing, but the number of cycles is considerably higher. The damage is characterized by a change in the energy dissipated per loading cycle. The point of failure is obtained from analyzing the variations in the dissipated energy per loading cycle using a parameter called dissipated energy ration DER. The proposed fatigue parameter is named N_{p20} and its application overcomes the inconveniences presented by the Superpave parameter G*sinδ.

Number of Loading Cycles

Figure 4.1 shows how binders that have similar characteristics in terms of G* in the first cycles of loading can present very different fatigue performance after many cycles of loading. During the NCHRP 9-10 project it was found that the best way to characterize the point when fatigue damage occurs is shown is by using the concept of change in the dissipated energy per cycle. The total energy dissipated per loading cycle (W_i) is defined in equation 2.

$$W_i = \pi \cdot \gamma_i \cdot \tau_i * \cdot \sin\delta \quad ; \quad \text{eq.2}$$

where W_i = total energy dissipated in loading cycle i

τ_i = stress applied

γ_i= strain applied

δ = phase angle

Equation 3 shows the accumulated dissipated energy after n cycles W_c.

$$W_c = \sum_{i=1}^{n} W_i \quad ; \quad \text{eq.3}$$

These two parameters (W_i and W_c) allow to define another parameter called Dissipated Energy Ratio (DER), which is determined according to equation 4.

$$DER = \frac{W_c}{W_n} \quad ; \quad \text{eq.4}$$

where W_n = energy dissipated at cycle n

A binder can be subjected to cyclic loading for a determined number of cycles without showing fatigue damage. During this stage, the energy dissipated per loading cycle remains constant. However, if the binder continues to be loaded and unloaded, a point is reached when it does not dissipate the same amount energy per cycle anymore. This is a sign of fatigue damage. This behavior is material specific and does not depend on the loading conditions. This behavior can be clearly shown using the DER. During the first cycles of loading, when W_n remains constant and W_c increases at a constant pace, so the DER shows a linear trend with 45° slope. When fatigue damage is reached the trend line followed by the DER depends on the mode of loading. In the strain controlled test the value of W_n decreases when fatigue damage is reached, so the DER increases. For the stress controlled test, however, the value of W_n increases when fatigue damage is reached, so the DER decreases. The variation in the DER versus cycles of loading for a typical binder under cyclic strain controlled and stress controlled loading are shown in Figure 4.2 and 4.3 respectively.

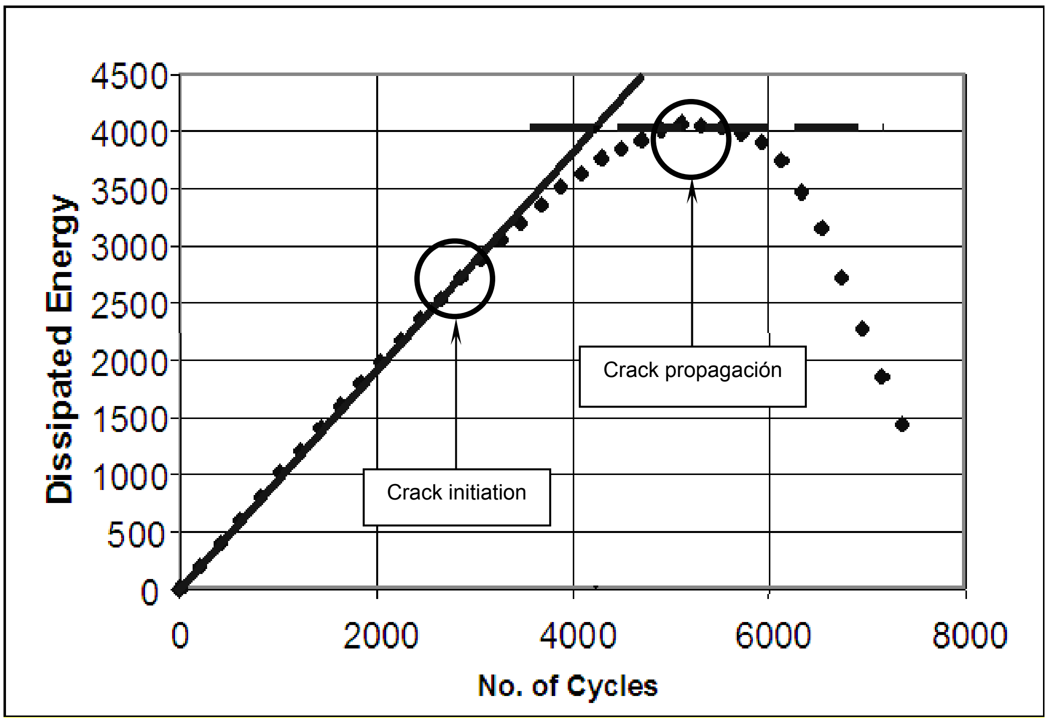


Figure 4.2 Variation in the DER for Stress Controlled Testing

The point where the trend line leaves the linear slope corresponds to the point where the fatigue cracks are initiating in the binder. This point is refereed as crack initiation point. However, after this point, if the binder is unloaded and enough resting time is given, it is still capable of recovering. After more cycles of testing the crack propagation point is reached. When the binder goes beyond this point no more healing is possible and the fatigue damage is not reversible. Conceptually, the fatigue life of the binder is represented by the number of cycles required to undergo the crack initiation without reaching the crack propagation.

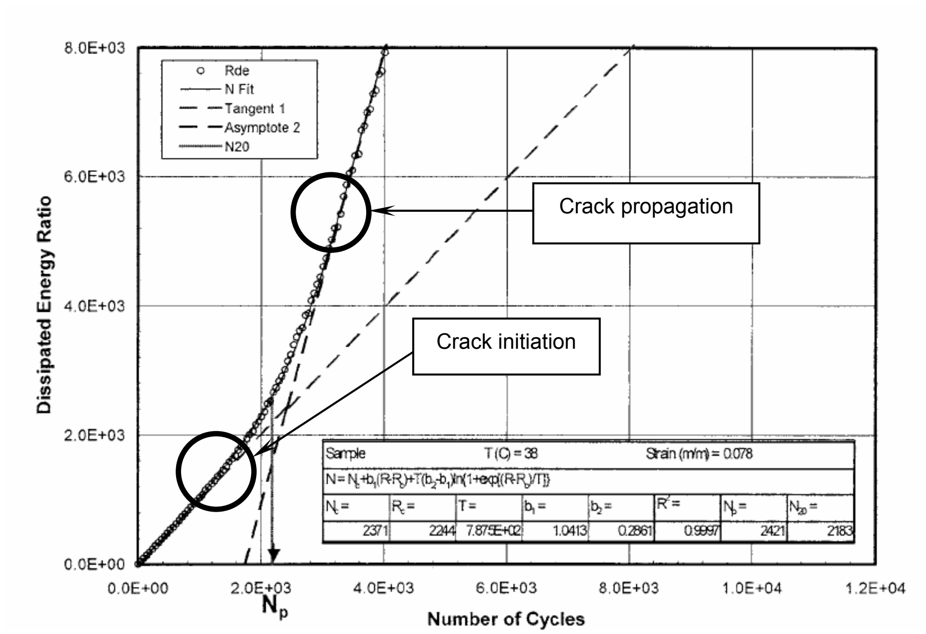


Figure 4.3 Variation in the DER for Strain Controlled Testing

Independence from the Mode of Loading

The fatigue damage point for binders, $N_{p20,}$ was defined as the point (cycle number n) where the dissipated energy ratio DER deviates 20% from the 45º inclination straight line. This point was selected because it provides independence of the mode of loading. If the initial input energy is the same, the N_{p20} value obtained using stress controlled or strain controlled test will be the same. The N_{p20} occurs after the crack initiation point and usually before the crack propagation point, so it provides a reasonable criterion for defining the fatigue failure of binders.

Stress controlled was selected as the most suitable test because it provides a more clear measurement of the Np20 value. It can be seen that both, the crack initiation and crack propagation are more clearly shown in the stress controlled testing shown in Figure 4.2 than in the strain controlled testing shown in Figure4.3.

Pavement Structure

Weak pavements deform more under traffic loads than strong pavements. This generates more energy input over the binder used in the weak pavement. Fatigue life of binders is highly sensitive to energy input. This fact is clearly shown in Figure 4.4, where the different fatigue lives shown by the same binder subjected to different stress levels is presented. This variable was considered in defining the new parameter N_{p20}. If the binder is going to be used in a weak structure, a high energy input is used in the testing. If the binder is for a strong structure, a lower energy input is considered for testing.

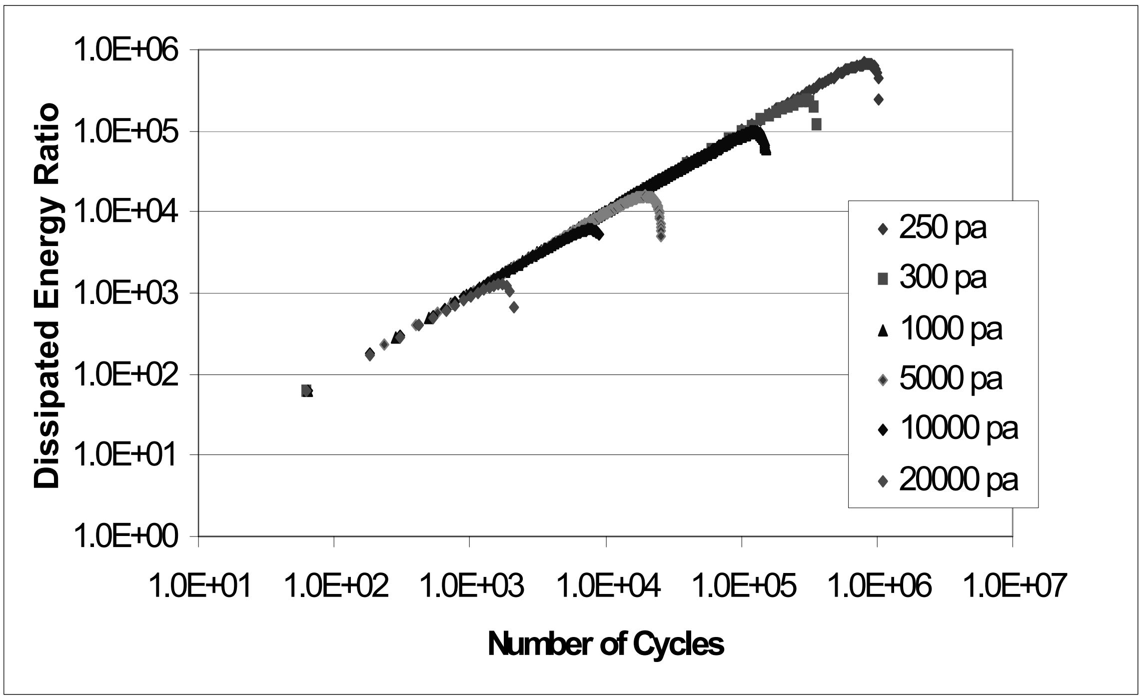


Figure 4.4 Fatigue life versus Stress Level Used in Testing

4.3 Determination of the Number of Cycles to Failure, Np20

The determination of the N_{p20} value depends mainly on two parameter: the energy input and temperature of testing. It has been proven that the fatigue life relationships vary with the temperature of the material. For each of the required pavement temperatures, the binder has to be tested at different energy levels for determining the fatigue relationships. When the energy concept is used, there is no need to test at different frequencies. The effect of frequency can be included in the energy input, as it will be explained later in this paper.

Fatigue Relationship

The relationship between the fatigue life of a binder (N_{p20}) and the energy input (W_i), for a specific pavement temperature, can be approximated by equation 5.

$$N_{p20} = K_1 \cdot (1/w_i)^{K2} \; ; \qquad \text{eq.5}$$

When plotted in a log-log graph, the relationship shown by equation 5 is linear. For determining the parameters K_1 and K_2 two points are needed. This means that, for a fixed pavement temperature, the binder has to be tested at two different energy levels in order to obtain the fatigue relationship. After the parameters are calculated, the fatigue relationship can be used to calculate the fatigue life at any input energy for the selected pavement temperature.

Energy Input

The two input energies needed to determine the fatigue relationship are chosen in order to represent the weak pavement and strong pavement conditions. Since there is not representative data available of the actual stress or strain that the binder would have in the actual pavement, some assumptions were made to select the input energies that represent each of the pavement structures. It was assumed that the binder in the strong pavement is loaded in the linear range. In the weak pavement, the binder is considered to be loaded in the non-linear range. For determining the energy needed to take the binder to the linear and non-linear regions, amplitude sweep testing was done. The amplitude sweep plot data for a typical binder is shown in Figure 4.5(a). The low strain/stress (strong pavement) was defined as ½ of the linear limit. The high strain/stress (weak pavement), was picked up slightly above the linear limit. Figure 4.5 (b) shows how the two energy input levels are used to construct the relationship between N_{p20} and the Wi.

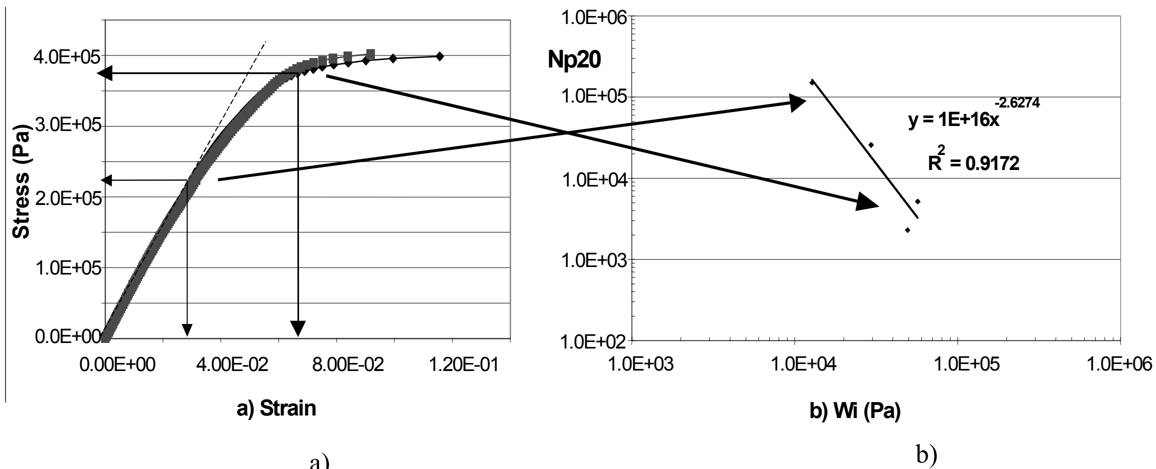


Figure 4.5 Amplitude Sweep Test Data Plot for a Typical Binder (a) and Np20 vs. W_i Relationship (b)

<u>Testing Temperatures</u>

Testing of fatigue life of asphalt binders has proved to be dependent on the testing temperature. Different fatigue relationships can be obtained for the same binder when different testing temperatures are used. For this reason, the temperature of testing has to be selected prior to the testing.

The testing temperature is selected as the average of the high and low pavement design temperatures of the PG grading. In the current AASHTO MP1, the temperature for checking the fatigue parameter G*·sinδ is chosen as the average plus 4°C. There is no need to add 4°C to the average temperature in the determination of N_{p20}, so the average should be used directly. For example for a PG 58-28 grade, the testing is conducted at 15°C for the new parameter, instead of the 19°C used in the current specifications. Since PG 58-28 is the binder used in the most populated areas of Wisconsin (south east region of the state), the average temperature of this PG grade was selected (15ºC) for the purpose of the present work. Besides this temperature, another temperature was also picked up for taking into account the fatigue that occurs in the pavement during the thaw season, when the structure under the pavement is its worse condition. The temperature selected for representing this condition, considering the climatic conditions of Wisconsin, was 6ºC.

Testing frequencies

It has been proven that the testing frequency is a factor to consider in the determination of the fatigue life of asphalt binders and mixtures. In fact, many studies conducted on asphalt mixtures suggested including the stiffness, which is a function of the frequency, as a parameter in the fatigue law. For this reason, the testing frequency has to be known prior to the testing.

The required frequency is related to the traffic speed. For 60 mph the testing frequency should be 10 Hz. For slow speed (15 mph) the testing frequency should be 2.5 Hz. Using the initial energy concept (W_i), the effect of frequency on fatigue life could be included in changing W_i so testing at multiple frequencies is not necessary. Equation 1 and equation 6 show the relationships for W_i in a strain controlled and stress controlled testing respectively.

$$W_i = \pi \cdot \tau_0^2 \cdot \frac{1}{G^*/\sin\delta}; \qquad \text{eq.6}$$

where: W_i = total energy dissipated in cycle i

τ_0 = maximum stress applied

G^* = complex modulus

δ = phase angle

The values of G* and δ vary with the testing frequency, so the effect of testing frequency can be included in the value of G* and δ for any of the modes of testing used. Considering this advantage the testing is done at 10 Hz only. This way the time required for testing is significantly reduced.

4.4 Testing to Derive Limits for N_{P20}

Test Information

A total of 18 modified asphalts and one base asphalt were tested using oscillatory shear measurements under stress controlled conditions. The change in properties was measured using the Dynamic Shear Rheometer (DSR) in accordance with the AASHTO Standard Procedures (TP5). Each asphalt was tested after aging with the Pressure Aging Vessel Oven (PAV). G* and Phase Angle (δ) measurements were used to compute the

Dissipated Energy Ratio, which in turn was used to compare the results for each binder under different testing conditions. Some information about the testing is listed below.

- Energy input: two levels of energy input were used. One in the middle of the linear range and one slightly over the linear limit, as explained earlier.
- Testing temperature was chosen as 15ºC, which corresponds to the average of the high and low temperature of the PG 58-28, as discussed earlier. An additional testing temperature was also chosen for the tested binders of 6°C which accounts for the freeze and thaw effects
- The testing frequency used was 10 Hz, which correspond to the fast traffic speed, as discussed earlier.

Summary of Binder Fatigue Testing Results

The results of 19 binders tested for fatigue performance are shown in Table 1.1. The measurements and calculations confirm the high dependency on the stress level of the binder fatigue life. The stress sensitivity was very similar for the two testing temperatures. For both, 15ºC and 6ºC, an average decrease in the fatigue life of 15 times was registered when the stress was changed from the linear to non linear range. However, for 6ºC the variations were higher. For this temperature the maximum decrease in fatigue life when moving from strong to weak structure was 108 times (D5) and the minimum was 2 times (B9). For 15ºC a maximum decrease of 39 (B5) times and a minimum of 5 times (D4) was shown.

From the test results, it is not possible to establish a clear trend in the temperature susceptibility of the fatigue life of binders. Some binders increase their fatigue life about 2 to 3 times (D6 and B9 for low stress; D5 high stress) when the testing temperature dropped from 15ºC to 6ºC. Other binders decrease their fatigue life more than 5 times (A1 for low stress; C5, B8, C4 and A1 for high stress) for the same temperature change.

The Superpave fatigue analysis of binders showed no correlation with the Np20 values. For example, the binders C5 (PG 58-28), B5 (PG 64-34) and D2 (PG 64-34) have the same average temperature of 15ºC. The three binders should have shown similar fatigue performances for the 15ºC testing temperature. The results however differ considerably, showing differences in the fatigue life up to 6 times between the each other.

Table 4.1 Results of Fatigue Tests at 6°C and 15°C (part I)

Binder Code	PG Grade	Testing Temp. [ºC]	Stress Level [kPa]	G* [Pa]	Phase Angle	Wi (Pa)	Measured Np20	K1	K2
B7	PG 58-40	15	183	3,579,750	49.0	22,189	40,056	-2.086	4.66E+13
	(Elvaloy)		137	4,481,980	46.3	9,519	234,056		
		6	430	12,177,900	43.5	32,849	22,906	-3.722	1.48E+21
			344	14,117,300	41.1	17,324	247,811		
B9	PG 58-34	15	288	6,224,400	49.9	31,888	10,615	-2.468	1.38E+15
	(Elvaloy)		200	7,200,010	46.9	12,737	102,213		
		6	659	24,331,600	40.0	36,008	28,094	-1.422	8.48E+10
			527	24,146,300	39.4	22,934	53,362		
D4	PG 58-34	15	371	10,979,500	46.5	28,550	23,217	-1.812	2.76E+12
	(SB)		253	12,316,500	44.3	11,395	122,655		
		6	861	37,057,300	36.8	37,666	19,700	-3.817	5.77E+21
			689	38,696,100	35.3	22,251	146,915		
D5	PG 64-40	15	161	2,131,640	51.9	30,075	4,363	-2.419	2.96E+14
	(SB)		97	2,833,290	48.0	7,757	115,665		
		6	336	7,682,110	46.2	33,338	2,046	-3.651	6.64E+19
			202	9,264,690	41.9	9,241	221,330		
C5	PG 58-28	15	615	20,454,400	42.2	39,048	12,843	-1.841	3.65E+12
	(Non		320	22,049,600	40.0	9,378	177,478		
	Modified)	6	984	61,116,800	31.1	25,679	82,213	-2.280	9.34E+14
			782	61,650,000	30.5	15,828	247,832		
B5	PG 64-34	15	322	5,202,160	51.8	49,230	7,294	-2.868	2.09E+17
	(Elvaloy)		198	6,645,800	48.0	13,762	282,148		
		6	630	20,747,400	42.6	40,683	27,518	-2.689	6.79E+16
			485	24,318,400	40.4	19,684	193,779		
D2	PG 64-34	15	420	11,139,200	46.3	35,981	44,157	-4.169	4.35E+23
	(SB)		336	11,736,100	45.0	21,379	386,842		
		6	840	32,186,400	38.1	42,482	114,939	-1.643	4.63E+12
			673	38,383,600	35.4	21,499	351,988		
A3	PG 64-28	15	568	16,036,800	45.3	44,954	9,159	-3.036	1.23E+18
	(SBS)		435	18,156,900	42.7	22,183	78,209		
		6	1264	44,836,900	43.3	76,810	16,532	-3.597	6.20E+21
			993	50,150,600	41.3	40,803	160,903		
B2	PG 64-28	15	570	18,142,200	39.8	36,050	16,052	-3.847	5.45E+21
	(Elvaloy)		395	20,151,500	37.3	14,751	499,513		
		6	1302	52,498,300	31.9	53,585	15,471	-5.074	1.54E+28
			1023	58,534,700	29.8	27,927	422,365		
D1	PG 64-28	15	733	25,272,600	39.4	42,366	12,592	-2.949	5.59E+17
	(SB)		500	27,706,800	37.0	17,050	184,492		
		6	1490	75,414,100	29.1	44,966	10,890	-4.347	1.84E+24
			1185	80,425,300	27.7	25,513	127,946		

Table 4.1. Results of Fatigue Tests at 6°C and 15°C (part II)

Binder Code	PG Grade	Testing Temp. [ºC]	Stress Level [kPa]	G* [Pa]	Phase Angle	Wi (Pa)	Measured Np20	K1	K2
D6	PG 70-34	15	289	5,193,520	52.4	40,006	16,836	-2.806	1.38E+17
	(SB)		208	6,578,040	48.9	15,582	237,300		
		6	650	17,503,000	44.8	53,447	10,560	-6.310	7.19E+33
			580	18,913,000	43.4	38,390	85,199		
B8	PG 70-34	15	336	6,491,070	49.1	41,289	8,805	-2.621	1.10E+16
	(Elvaloy)		210	8,006,560	45.3	12,305	210,136		
		6	620	24,566,800	39.5	31,275	50,767	-5.874	1.29E+31
			540	24,651,400	39.1	23,461	274,776		
C4	PG 64-22	15	880	37,044,500	36.6	39,182	21,115	-2.804	1.60E+17
	(SBS)		609	39,899,600	34.5	16,548	236,745		
		6	1390	101,987,000	25.8	25,897	132,582	-1.966	6.27E+13
			973	114,387,000	26.6	11,651	637,255		
C2	PG 70-28	15	563	15,833,400	43.5	43,201	11,307	-3.659	1.04E+21
	(SBS)		450	19,121,900	40.5	21,597	142,928		
		6	1355	52,608,500	33.2	59,964	12,569	-2.784	2.52E+17
			996	58,071,200	30.6	27,349	111,818		
A1	PG 70-28	15	573	18,360,400	41.0	36,891	4,076	-3.670	2.34E+20
	(SBS)		458	19,725,400	39.2	21,113	31,598		
		6	1141	56,841,100	30.6	36,576	25,538	-4.283	8.94E+23
			913	57,905,400	29.4	22,222	215,817		
B4	PG 70-28	15	545	15,813,200	41.8	39,302	11,278	-3.935	1.35E+22
	(Elvaloy)		436	16,959,900	40.1	22,675	98,204		
		6	1251	50,962,900	32.6	52,044	7,846	-41.487	3.65E+199
			1042	44,256,000	38.8	48,243	182,420		
B6	PG 76-34	15	295	5,421,190	49.6	38,413	21,671	-2.276	5.90E+14
	(Elvaloy)		236	6,591,480	46.5	19,255	104,370		
		6	675	23,693,500	39.7	38,593	15,579	-2.843	1.70E+17
			540	24,320,400	38.9	23,668	62,545		
C6	PG 76-28	15	509	14,148,100	43.4	39,518	4,613	-4.597	6.27E+24
	(SBS)		407	17,247,600	41.0	19,813	110,277		
		6	1097	44,855,800	34.0	47,097	10,050	-4.896	7.61E+26
			894	47,040,900	32.3	28,530	116,945		
B3	PG 76-28	15	588	16,129,900	42.2	45,202	9,475	-3.772	3.45E+21
	(Elvaloy)		452	17,506,400	39.6	23,387	113,804		
		6	1257	52,291,800	32.4	50,911	29,039	-3.352	1.74E+20
			898	57,289,100	30.2	22,273	464,058		

The measured values of G* and δ shown very little variations with the stress level. The maximum variations were around 20% for G* and 30% for δ. No trend could be established, since for some binders the value of G* or δ increased with increasing the stress level, and for other binder the value of the parameters decrease.

All the binders, except B4 (PG 70-28) for 6ºC, showed reasonable values for the parameters K_1 and K_2. These parameters were replaced in equation 5 to calculate the fatigue lives of the binders for different levels of initial energy, as it will be shown later in this paper. For all the cases, but the one mentioned, the fatigue lives obtained were reasonable, within the expected values. Only for B4 at 6ºC the fatigue relationship shown to be unrealistic, forecasting very high fatigue lives that are not likely to be possible. This could have been generated by errors in the binder testing. For this reason, the testing results for B4 at 6ºC were excluded from the analysis carried out in the following sections.

4.5 Computation of Np20 for Each Traffic Speed

Fast Traffic Speed

The values measured for the binder parameters shown in Table 4.1 were measured with a frequency of 10 Hz, which corresponds to fast traffic speed. The Np20 values shown in Table 4.1 are the ones calculated using the high and low input energy of the corresponding binder. As explained earlier, amplitude sweep testing was carried out to select the W_i needed to load the binder in the linear range (strong structure) or nor linear range (weak structure). That's why each binder has it's own value of W_i for strong structure and weak structure, as it is shown in the 7th column of Table 4.1. However, the energy input is a function of the pavement structure only (when the load is fixed), so a single value of W_i has to be selected for each of the pavement structures. To estimate a representative W_i value for each pavement structure, the input energies of all the binders were averages for the corresponding pavement structure (for each temperature). Table 4.4 shows the average values of the input energy for each combination of parameters.

Table 4.2 Selected Wi Values for Strong and Weak Structure, Fast Traffic

Temperature	Average Wi Values [Pa]	
	Strong (Linear Range)	Weak (Non Linear Range)
15 ºC	16431	38070
6 ºC	23364	42657

The values shown on Table 4.2 were rounded. Wi values of 25000 Pa and 45000 Pa were used for estimating Np20 at 6°C of the strong structure and the weak structure, respectively. For 15°C, the Wi values used were 15000 Pa and 40000 Pa for the strong structure and weak structure, respectively. The estimated Np_{20} using these four rounded Wi values are shown on the sixth column of Table 4.5. The Np20 values were calculated using the relationship shown in equation 6 and the K_1 and K_2 values presented in Table 4.1. It should be mentioned here that if a better estimation of strain or stress in a pavement structure could be determined, it could be used to estimate Np_{20}. If these values are not available, the values suggest here could be used.

Slow Traffic Speed

For slow traffic speed, the Wi values have to be corrected for including the effect of the lower testing frequency. The lower frequency would result in variations in the G* and δ values. Equation 5 shows the relationship between W_i, G* and δ for a stress controlled test. For accounting the variation of W_i due to testing frequency, an estimation of the variation of the parameter G*/sinδ with respect to frequency is needed. It has been shown that at the intermediate pavement temperature, the value of G*·sinδ decreases approximately 7 times, and the G* decreases around 8 times when the frequency of testing is lowered from 10 hz to 0.1 hz (1% strain). Using this two parameters, the decrease in G*/sinδ can be estimated in approximately 10 times for the same change in frequency. This relationship was used to approximate the variations in G*/sinδ when the frequency is changed from 10 Hz to 2.5 Hz. Considering linear interpolation, the parameter would decrease approximately 30% when the frequency decreases 4 times. Table 4.3 shows the values of G*/sinδ for low speed, which correspond to 70% of the value at high speed. These values were used to calculate the corresponding Wi values for low speed, shown in the 7^{th} column of Table 4.3.

Table 4.3 Wi Values for Slow Traffic (part I)

Binder Code	PG Grade	Testing Temp. [ºC]	Stress Level [kPa]	G/sinδ [Pa] (60 mph)	G/sinδ [Pa] (15 mph)	Wi (15 mph)
B7	PG 58-40 (Elvaloy)	15	183	4743319	3320323	31685
			137	6199565	4339696	13587
		6	430	17691736	12384215	46904
			344	21475774	15033042	24729
B9	PG 58-34 (Elvaloy)	15	288	8137483	5696238	45585
			200	9861058	6902741	18204
		6	659	37854179	26497925	51487
			527	38042753	26629927	32763
D4	PG 58-34 (SB)	15	371	15136666	10595666	40809
			253	17635335	12344735	16289
		6	861	61864405	43305083	53778
			689	66966484	46876538	31814
D5	PG 64-40 (SB)	15	161	2708845	1896191	42944
			97	3812653	2668857	11075
		6	336	10643819	7450673	47601
			202	13873107	9711175	13200
C5	PG 58-28 (Non Modified)	15	615	30451486	21316040	55742
			320	34303930	24012751	13397
		6	984	118324256	82826979	36724
			782	121471888	85030322	22593
B5	PG 64-34 (Elvaloy)	15	322	6619868	4633908	70291
			198	8943005	6260103	19674
		6	630	30652432	21456703	58111
			485	37522369	26265658	28134
D2	PG 64-34 (SB)	15	420	15407967	10785577	51380
			336	16597736	11618415	30526
		6	840	52164236	36514965	60705
			673	66262440	46383708	30676
A3	PG 64-28 (SBS)	15	568	22562158	15793511	64173
			435	26774421	18742095	31717
		6	1264	65378770	45765139	109672
			993	75987404	53191183	58237
B2	PG 64-28 (Elvaloy)	15	570	28343026	19840118	51445
			395	33254775	23278343	21056
		6	1302	99348744	69544121	76577
			1023	117785382	82449767	39875

Table 4.3 Wi Values for Slow Traffic (part II)

Binder Code	PG Grade	Testing Temp. [ºC]	Stress Level [kPa]	G/sinδ [Pa] (60 mph)	G/sinδ [Pa] (15 mph)	Wi (15 mph)
D1	PG 64-28	15	733	39817251	27872076	60559
	(SB)		500	46039895	32227926	24369
		6	1490	155070210	108549147	64251
			1185	173021105	121114773	36423
D6	PG 70-34	15	289	6555214	4588650	57180
	(SB)		208	8729437	6110606	22242
		6	650	24840415	17388291	76332
			580	27526984	19268889	54845
B8	PG 70-34	15	336	8587924	6011547	58997
	(Elvaloy)		210	11264421	7885095	17570
		6	620	38623247	27036273	44666
			540	39088245	27361772	33480
C4	PG 64-22	15	880	62133335	43493334	55934
	(SBS)		609	70445254	49311678	23628
		6	1390	234334821	164034375	37003
			973	255472487	178830741	16631
C2	PG 70-28	15	563	23002351	16101646	61732
	(SBS)		450	29444028	20610820	30865
		6	1355	96079999	67255999	85760
			996	114082558	79857790	39025
A1	PG 70-28	15	573	27986576	19590603	52650
	(SBS)		458	31210401	21847281	30163
		6	1141	111665991	78166193	52323
			913	117959895	82571927	31714
B4	PG 70-28	15	545	23725147	16607603	56185
	(Elvaloy)		436	26330839	18431587	32400
		6	1251	94593543	66215480	--
			1042	70630115	49441081	--
B6	PG 76-34	15	295	7118890	4983223	54862
	(Elvaloy)		236	9087211	6361047	27506
		6	675	37093420	25965394	55125
			540	38729991	27110994	33789
C6	PG 76-28	15	509	20591896	14414327	56465
	(SBS)		407	26290346	18403242	28277
		6	1097	80217334	56152134	67326
			894	88035801	61625061	40743
B3	PG 76-28	15	588	24013387	16809371	64616
	(Elvaloy)		452	27464971	19225480	33384
		6	1257	97593429	68315400	72659
			344	21475774	15033042	24729

The Wi values shown in Table 4.3 are different for each of the binders. A single Wi value has to be selected for each of the pavement structures, as explained earlier. Following the same procedure used for high speed, the Wi values for slow speed were estimated from the average value of all the binders at the corresponding pavement structure and temperature. Table 4.4 shows these average values of the input energy for each combination of parameters.

Table 4.4 Selected Wi Values for Strong and Weak Structure, Slow Traffic

Temperature	Estimated Wi Values	
	Strong (Linear Range)	Weak (Non Linear Range)
15 C	23470	54381
6 C	33358	60945

The values shown on Table 4.4 were rounded. Wi values of 35000 Pa and 60000 Pa were used in estimating Np20 at 6°C for the strong structure and the weak structure, respectively. For 15°C, the Wi values used were 25000 Pa and 55000 Pa for strong structure and weak structure, respectively. The estimated Np_{20} using these four values of Wi are shown on the last column of Table 4.5. The Np20 values were calculated using the relationship shown in equation 6 and the K_1 and K_2 values presented in Table 4.1.

Table 4.5 Estimated Np20 Values for Fast and Slow Traffic (Part I)

Binder Code	PG Grade	Testing Temp. [ºC]	Pavement Structure	Np20 at Selected Wi Values (60 mph)	Np20 at Selected Wi Values (15 mph)
B7	PG 58-40	15	Weak	11718	6031
			Strong	90649	31233
		6	Weak	7099	2434
			Strong	63282	18090
B9	PG 58-34	15	Weak	6067	2765
			Strong	68270	19353
		6	Weak	20461	13591
			Strong	47202	29252
D4	PG 58-34	15	Weak	12601	7076
			Strong	74533	29533
		6	Weak	9989	3331
			Strong	94179	26071
D5	PG 64-40	15	Weak	2189	1013
			Strong	23469	6822
		6	Weak	684	239
			Strong	5851	1713
C5	PG 58-28	15	Weak	12286	6836
			Strong	74751	29187
		6	Weak	22876	11871
			Strong	87394	40575
B5	PG 64-34	15	Weak	13230	5308
			Strong	220389	50928
		6	Weak	20983	9682
			Strong	101898	41238
D2	PG 64-34	15	Weak	28398	7529
			Strong	1694784	201486
		6	Weak	104562	65172
			Strong	274699	158026
A3	PG 64-28	15	Weak	13056	4965
			Strong	256583	54401
		6	Weak	113138	40196
			Strong	937287	279398
B2	PG 64-28	15	Weak	10760	3160
			Strong	468359	65629
		6	Weak	37525	8716
			Strong	740781	134329
D1	PG 64-28	15	Weak	14918	5832
			Strong	269192	59668
		6	Weak	10854	3108
			Strong	139758	32367

Table 4.5 Estimated Np20 Values for Fast and Slow Traffic (Part II)

Binder Code	PG Grade	Testing Temp. [ºC]	Pavement Structure	Np20 at Selected Wi Values (60 mph)	Np20 at Selected Wi Values (15 mph)
D6	PG 70-34 (SB)	15	Weak	16843	6892
			Strong	264050	62977
		6	Weak	31267	5090
			Strong	1275960	152679
B8	PG 70-34 (Elvaloy)	15	Weak	9568	4153
			Strong	125057	32790
		6	Weak	5989	1105
			Strong	189186	26212
C4	PG 64-22 (SBS)	15	Weak	19926	8159
			Strong	311808	74439
		6	Weak	44753	25424
			Strong	142094	73342
C2	PG 70-28 (SBS)	15	Weak	14986	4673
			Strong	542460	83674
		6	Weak	27952	12548
			Strong	143580	56269
A1	PG 70-28 (SBS)	15	Weak	3029	941
			Strong	110779	16996
		6	Weak	10511	3066
			Strong	130311	30840
B4	PG 70-28 (Elvaloy)	15	Weak	10523	3006
			Strong	499177	66884
		6	Weak	--	--
			Strong	--	--
B6	PG 76-34 (Elvaloy)	15	Weak	19763	9573
			Strong	184259	57606
		6	Weak	10067	4444
			Strong	53530	20568
C6	PG 76-28 (SBS)	15	Weak	4363	1009
			Strong	396388	37861
		6	Weak	12561	3071
			Strong	223267	42992
B3	PG 76-28 (Elvaloy)	15	Weak	15027	4520
			Strong	607824	88489
		6	Weak	43920	16743
			Strong	315076	101988

4.6 Derivation of New Specification Criteria

Mechanistic Binder Specification Framework

The mechanistic approach for fatigue resistance should incorporate the following parameters: traffic speed, traffic volume, pavement structure and temperature. As discussed earlier, these parameters are essential to characterize the fatigue life of binders. Table 4.6 depicts a specification framework in which the parameters mentioned are explicitly included in the grading system. In the following sections an example of how such a system can be implemented in Wisconsin is presented.

Table 4.6 Example of a Mechanistic Binder Specification Framework for Fatigue Resistance that Includes Pavement Design Temperature, Traffic Conditions and Pavement Structure

Purpose	Test Parameter	AASHTO Method	Testing Rate	Testing Stress	Testing Temp ºC	**Criteria**	**Traffic Level (Millions ESALs)**		
							L	**M**	**H**
							<1.0	**1.0-3.0**	**>3.0**
Test on RTFO and PAV Aged Binder									
Fatigue Resistance	Cyclic stress time sweep	TP5	Loading rate (rad/s)	Stress amplitude strong/ weak			**Np20**		
Traffic Speed: Fast			100	50 kPa / 100 kPa	IT	**Minim.**	**(g)**	**(h)**	**(i)**
Traffic Speed: Slow			10	50 kPa / 100 kPa	IT	**Minim.**	**(g)/10**	**(h)/10**	**(i)/10**

Note: In the best judgement of NCHRP9-10, the following limits are recommended:
(g) = 5000, (h) = 15000, (i) = 45000

Traffic and Weather Information

The temperatures selected for testing are 15ºC and 6ºC as discussed earlier. 15ºC is selected to characterize the cumulated fatigue damage that would occur in the pavement during the complete spring season. 15ºC is very close to the average spring temperatures in south east Wisconsin, the most populated area of the state. Spring is a critical season for fatigue analysis due to the decreasing in the soil bearing capacity after the thaw season.

6ºC is chosen to take into account the fatigue damage induced in the pavement during the thaw season itself. 6ºC represents approximately the temperatures in the pavement for the thaw season in south east Wisconsin. Thaw season is important to consider since bigger stresses are induced in the pavement in this period. The thaw effect provokes deterioration in the bearing capacity of the structure underneath the asphalt layers so the deformations are higher under the same traffic loads.

The six levels of traffic volume recognized on Wisconsin highways were used for deriving the fatigue specifications. The traffic levels considered are shown in the second column of Table 4.10, in accordance to the Wisconsin PG Binder Selection Criteria.

Two different traffic speeds were considered. High speed equal to 60 mph (10 Hz testing frequency), which is 5 miles per hour lower than the maximum allowable traffic speed in Wisconsin. Low speed equal to 15 mph (2.5 Hz testing frequency), for taking into account the slow movements of traffic in urban areas.

Deriving Field Conversion Factors

The approach used for deriving the field conversion factors for fatigue is different from the one used for rutting. The current recommendations for the selection of binders specify the high temperature of the PG grading suitable for each level of traffic. These specifications are focused mainly in the rutting performance. This was a big advantage for determining the rutting field factors. It allowed relating traffic volumes with a specific high temperature PG grade.

However, for the fatigue there is not a direct way to relate the binder performance with the expected pavement life. A new approach was selected for this distress. The field correlation factors were obtained considering the fatigue performance only. Specifically, the relationship between the binder fatigue performance and the expected life of the binder in service was developed relating the N_{p20} binder values with the traffic levels. The worst performing binders were assigned to the lower levels of traffic volume and the best performing binders were assigned to the higher volumes of traffic. In order to simplify the derivation of the field conversion factors, only four levels of traffic were used, instead of the six levels that will be used in the final specifications. The four levels of traffic considered are shown in column 4 of Table 4.8. This procedure, that could seem arbitrary, has a logical foundation. The binders tested are binders that have been used in the construction roads in Wisconsin, so they meet the current specifications. What it is done in the present analysis is reorganizing the binders and assigning them to each traffic category considering the fatigue performance in terms of N_{P20}. On the other hand, this was found to be the only available way to relate binder performance to pavement life. Further field studies are recommended to validate the assumptions presented here and to obtain more accurate relationships between laboratory and field performance.

When determining the field correlation factor between Np_{20} and ESALs, it cannot be assumed that the pavement structure will be strong of weak. Assuming a weak pavement structure would be too conservative, especially for the high volume roads. Considering a strong pavement structure would be not very safe for the low volume roads. There is not a perfect answer for this problem. It was decided that the best alternative was considering an average pavement structure for the calculation of the field correlation factors. An average pavement structure means a structure where the binder would be subjected to an average energy level under the traffic loads. The average energy level (W_{ia}) was selected as the mean value between the weak (W_{iw}) and strong (W_{is}) energy levels, as shown in equation 7.

$$W_{ia} = \frac{W_{iw} + W_{is}}{2} \; ; \qquad \text{eq.7}$$

The traffic speed assumed for determining the correlation factors is the standard highway speed, which corresponds to the high speed defined in the present work. The high traffic speed corresponds to the standard speed to most of the road projects, so it was assumed to be more representative. Also, deriving the field validation factors from the high speed is more conservative than using the slow speed. The required N_{P20} values are higher when the fast speed is used to derive them. Using the values of W_{iw} and W_{is} shown in Table 4.4, the values for W_{ia} for 6ºC and 15ºC were obtained. The calculated values were:

- W_{ia} 6ºC = 33010 [Pa]
- W_{ia} 15ºC = 27251 [Pa]

The values shown above were rounded to 35000 Pa and 25000 Pa for 6ºC and 15ºC respectively. The corresponding N_{P20} values were calculated by using the correlation shown in equation 5 and the K_1 and K_2 values for each binder shown in Table 4.1. Table 4.7 shows the Np_{20} values for each of the binders using the average energy input.

The binders were divided into four groups according to their Np_{20} values, for each of the testing temperatures. The four groups are shown in Table 4.8. The fourth column of Table 4.8 shows the range of N_{p20} values for each group. The binders with the lower N_{p20} values were assigned to the lower levels of traffic. The binders with the higher N_{p20} values were related to the higher volumes of traffic. The levels of traffic used for this case were the same previously used for rutting. The four levels of traffic volume considered here are shown in the fourth column of Table 4.8.

Once the binders were grouped, the average Np20 value was calculated for each group and for each temperature. The average Np20 value of the group was related to the corresponding level of traffic volume. Table 4.9 shows the Np_{20} values for each of the binder groups and for each of the temperatures.

The correlation between the group Np20 values and the corresponding ESALs shown to be good for a power relationship. Figure 4.6 shows the plots of the Np20 versus ESALs. The correlation coefficients are equal to 0.98 and 0.89 for 15ºC and 6ºC respectively.

Table 4.7 Np_{20} Values for Average Pavement Structure

Binder Code	PG Grade (Modification)	Testing Temp. [ºC]	Estimated Np20 at Average Wi
B7	PG 58-40 (Elvaloy)	15	31233
		6	18090
B9	PG 58-34 (Elvaloy)	15	19353
		6	29252
D4	PG 58-34 (SB)	15	29533
		6	26071
D5	PG 64-40 (SB)	15	6822
		6	1713
C5	PG 58-28 (Non Modified)	15	29187
		6	40575
B5	PG 64-34 (Elvaloy)	15	50928
		6	41238
D2	PG 64-34 (SB)	15	201486
		6	158026
A3	PG 64-28 (SBS)	15	54401
		6	279398
B2	PG 64-28 (Elvaloy)	15	65629
		6	134329
D1	PG 64-28 (SB)	15	59668
		6	32367
D6	PG 70-34 (SB)	15	62977
		6	152679
B8	PG 70-34 (Elvaloy)	15	32790
		6	26212
C4	PG 64-22 (SBS)	15	74439
		6	73342
C2	PG 70-28 (SBS)	15	83674
		6	56269
A1	PG 70-28 (SBS)	15	16996
		6	30840
B4	PG 70-28 (Elvaloy)	15	66884
		6	--
B6	PG 76-34 (Elvaloy)	15	57606
		6	20568
C6	PG 76-28 (SBS)	15	37861
		6	42992
B3	PG 76-28 (Elvaloy)	15	88489
		6	101988

Table 4.8 Grouping of Binders and Traffic Levels for Fatigue Field Conversion Factors

Group Number	Binder Grouping 15ºC	Binder Grouping 6ºC	Range of Values for Np_{20}	Traffic Level
1	B9 (PG 58-34) D5 (PG 64-40) A1 (PG 70-28)	B7 (PG 58-40) D5 (PG 64-40) B6 (PG 76-34)	0 – 25,000	500,000
2	B7 (PG 58-40) D4 (PG 58-34) C5 (PG 58-28) B8 (PG 70-34) C6 (PG 76-28)	B9 (PG 58-34) D4 (PG 58-34) D1 (PG 64-28) B8 (PG 70-34) A1 (PG 70-28)	25,000 – 40,000	5,000,000
3	B5 (PG 64-34) A3, B2, D1 (PG 64-28) D6 (PG 70-34) B4 (PG 70-28) B6 (PG 76-34)	C5 (PG 58-28) B5 (PG 64-34) C2 (PG 70-28) C6 (PG 76-28)	40,000 – 70,000	20,000,000
4	D2 (PG 64-34) C4 (PG 64-22) C2 (PG 70-28) B3 (PG 76-28)	D2 (PG 64-34) A3, B2 (PG 64-28) C4 (PG 64-22) D6 (PG 70-34) B3 (PG 76-28)	+ 70,000	50,000,000

Table 4.9 Np_{20} Values for Each Binder Group and Temperature

ESALs [Millions]	Average ESALs	Binder Group	Ave. Np_{20} 15ºC	Ave. Np_{20} 6ºC
0 - 1	500000	1	14390	13457
1 - 10	5000000	2	32121	28948
10 - 30	20000000	3	59727	45268
30 +	50000000	4	112022	149960

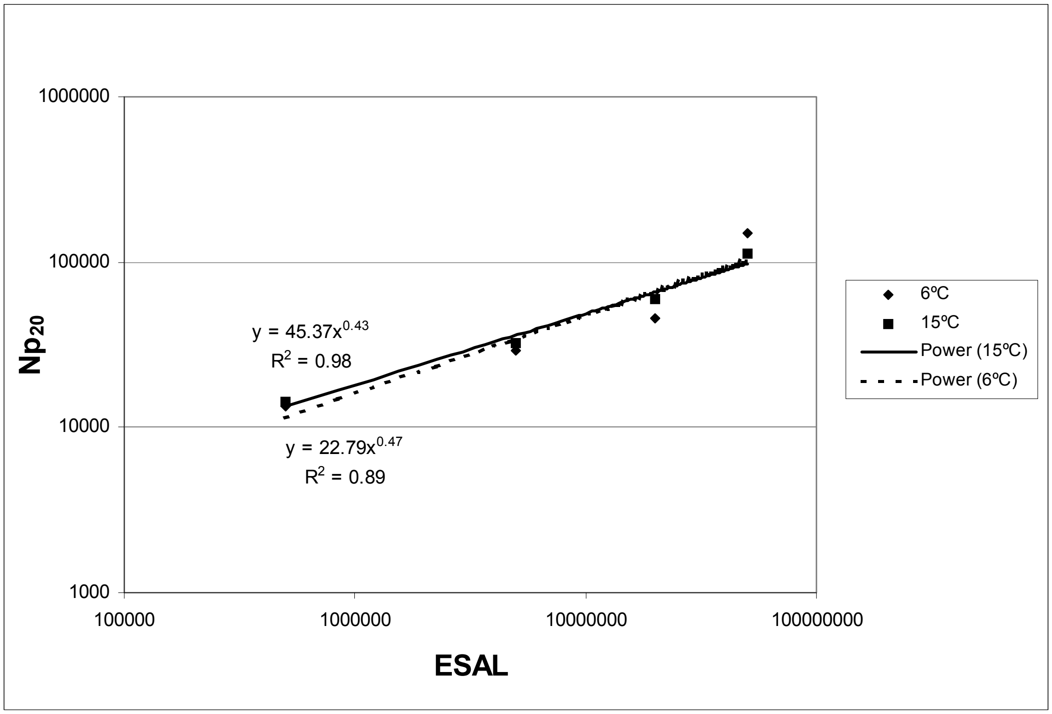


Figure 4.6 Np20 versus Allowable ESALs, Average Pavement Structure

The field conversion factors are represented by the slope of the trend lines shown in the Figure 4.6. It can be seen that there is as slightly different trend for each of the temperatures. The Np20 values for 6ºC are a little lower than the ones at 15ºC for the low volume traffic. For the high volume traffic, however, the Np20 is a little bit higher for 6ºC than for 15ºC.

4.7 Binder Criteria for Fatigue and Ranking of Binders

The proposed binder fatigue criteria was derived using the relationships shown in Figure 4.6. A minimum Np20 value was obtained for each traffic category. The required Np20 does not depend neither on the pavement structure nor on the traffic speed. Since Np20 is related to the number of repetitions that the pavement will have to support during its service life, the Np20 depends on the traffic volume only.

The required Np_{20} values were obtained by operating the average volume of each traffic range by the corresponding field conversion factor (15ºC or 6ºC). For example, the minimum value of Np20 specified for the traffic range from 1 to 3 millions ESALs is the one calculated using 2 million ESALs. Columns 4th and 5th of Table 4.10 show the specified Np_{20} values for each level of traffic volume and speed.

Once the required Np_{20} values are known for a determined level of traffic, the applicable binders can be obtained for each pavement structure. For each binder, the Np_{20} at the corresponding pavement structure, pavement temperature and traffic speed has to be compared with the required Np_{20}, at the selected traffic level. The applicable binders for each condition are shown in columns 6th to 9th of Table 4.8. For example, lets determine if B5 (PG 64-34, elvaloy) is suitable for 15ºC, strong structure, slow traffic speed and 20 million ESALs traffic volume. The Np20 of B5 for 15ºC, strong structure and slow traffic speed is equal to 50928 (Table 4.5). The minimum Np20 required for 20 million ESALs, 15ºC and slow traffic speed is 62548 (Table 4.10), so B5 does not satisfy the requirements. It means that according to the proposed criteria, B5 is will not have 20 million ESALs service life (fatigue life) under such pavement conditions and traffic speed. Column 8 of Table 4.10 (slow speed) shows B5 in the 3 – 10 million ESALs range. This means that the binder, under the mentioned conditions, can be used for any traffic volume up to 10 million ESALs without experiencing significant fatigue damage.

The results reflect clearly how the pavement structure affects the performance of the binders. For each temperature and traffic speed, when one binder is applicable for a determined traffic volume category for strong structure, then the same binder is applicable for a traffic volume between 3 and 5 categories lower for weak structure. This trend can be verified in Table 4.10 for most of the binders by comparing the position in which they are in column 6 (or 8) respect to the position they are in column 7 (or 9).

The ranking of the binders is similar for both of the temperatures considered. In both cases, the binders show similar variations in categories with changing from strong to weak structure. However, for 6ºC and weak structure, it can be observed that more binders are suitable for higher traffic levels that for 15ºC and weak structure. None of the binders seem to be adequate for traffic volumes higher than 3 million ESALs for 15ºC and weak structure. However, for 6ºC and weak structure, it is possible to find some binders applicable until 30 million ESALs or more (D2).

The variations with traffic speed follow the expected trend for both structures and temperatures. In each case, a binder applicable for a determined traffic category under fast traffic is applicable for a traffic level 1 to 3 categories lower for low speed. This can be visualized comparing the position where the binders are in the upper part (fast speed) respect to the lower part (slow speed) of columns 6 to 9 in Table 4.10.

Table 4.10 Binder Criterion for Fatigue and Ranking of Binders

Speed	Volume Range [million ESAL]	Average Volume [million ESAL]	Minimum Np20		Applicable Binders 15ºC		Applicable Binders 6ºC	
			15 [ºC]	6 [ºC]	Strong Structure	Weak Structure	Strong Structure	Weak Structure
Fast (60 mph)	0 - 0.3	0.15	7629	6173		D4, B2, B5, B7, C5, B4, A3, B8		B7, D1, A1, D4, B6
	0.3 - 1.0	0.65	14332	12297		C4, D1, D6, B3, B6, C2		C6, B9
	1.0 - 3.0	2	23239	20856	D5	D2		C2, B5, C5, D6
	3.0 - 10	6.5	38576	36292			B9, B6	B2, B3, C4
	10.0 - 30.0	20	62548	61550	B7, B9, D4, C5		B7, C5, D4	
	> 30.0	50	92753	94680	B5, A3, D1, B8, A1, C6, D6, C4, C2, B4, B6, B3, B2, D2		B5, A3, D1, B8, A1, C6, D6, C4, C2, B4, B3, B2, D2	D2, A3
Slow (15 mph)	0 - 0.3	0.15	7629	6173		C4, B6		B5, B2, C5
	0.3 - 1.0	0.65	14332	12297	B9, A1		B7	B9, C2, B3
	1.0 – 3.0	2	23239	20856	D4, C5, B7, B8, C6		B9, D4, B8, B6, D1, A1	C4
	3.0 - 10	6.5	38576	36292	B5, A3, D1, B6		C6, B5, C5, C2	A3
	10.0 - 30.0	20	62548	61550	D6, C4, C2, B4, B3, B2		C4	D2
	> 30.0	50	92753	94680	D2		B3, D6, A3, B2, D2	

NOTE: The table presents only the maximum traffic volume that the binders can resist for the different conditions. The binders are applicable for any traffic level lower or equal to the one presented in the table for the corresponding pavement conditions and traffic speed.

A general trend can be observed with respect of the hardness of the asphalt and the fatigue performance according to the proposed criteria. It seems that the softer asphalts have lower fatigue life than the harder ones. This is clearly evidenced for strong structure, 15ºC and fast speed. In this case, all the binders that are not applicable for the highest traffic category are the softer binders of the tested universe (PG 58-40, PG 58-34, PG 64-40, PG 58-28). This is tendency is contrary to the Superpave criteria, where it is assumed that softer asphalts would have better fatigue performance than harder asphalts, for a given testing temperature.

4.8 Proposed Binder Specifications for Fatigue

The proposed specification criteria for fatigue was derived from Table 4.10. The calculated Np20 values were rounded before being included in the specification. Table 4.11 shows the proposed specification for the Wisconsin state, which follows the framework presented before in Table 4.6. It provides a framework for selecting a binder suitable for a specific traffic speed, traffic volume, pavement structure and pavement temperature.

Table 4.11 Proposed Specifications for Fatigue Performance of Binders

Binder Requirements for Fatigue Resistance	Input Energy		Testing Temperature	Traffic Level (Millions ESALs)					
				0 - 0.3	0.3 - 1	1 - 3	3 -10	10 - 30	> 30
Test on RTFO and PAV Aged Binder									
Traffic Speed [mph]	Strong [Pa]	Weak [Pa]	[ºC]	Minimum Np_{20}					
60	15000	40000	15	7500	15000	25000	40000	65000	95000
	25000	45000	6	6000	12000	20000	35000	60000	95000
15	25000	55000	15	7500	15000	25000	40000	65000	95000
	35000	60000	6	6000	12000	20000	35000	60000	95000

It should be mentioned again that the testing is always carried out at 10 Hz, independently of the traffic speed of the projected road. Only the input energy is changed for including the speed effect, as explained earlier.

4.9 Summary of Findings for the Binder Fatigue Study

- The results obtained for the ranking of binders according to the proposed criteria are reasonable. The binders show better performance for strong structure than for weak structure. This was expected since in strong structures the failures are often due to rutting instead of fatigue (or low temperature cracking). Also, the binders show better results when the traffic is fast than when the traffic is slow.
- Considering the traffic speed by changing the input energy only is a very convenient approach. Since all the testing is done at high frequency (10 Hz), the time of testing is reduced considerably.
- The fatigue resistance of binders, as measured by the Np20 values, shows a great disparity in fatigue performance for binders of the same PG grade. This is another proof that the PG grade is not a good indicator of binder contribution for fatigue resistance in asphalt mixtures. The binder fatigue results show a significant effect of temperature and pavement structure for these binders.
- The proposed limits are derived based on assumptions and approximations that may or may not be completely valid. The assumption that for the strong structure the binder perform in the linear range and for weak structure in the non linear range might not be very accurate. If a better approximation of the real stress (strain) in the pavement is known, it should be used for determining the initial input energy.
- The proposed criteria and limits are considered as a starting point for further evaluation by the State Highway Agencies and the Industry. Research is carried out currently in order to validate the field conversion factors assumed in this work.

CHAPTER 5: LOW-TEMPERATURE CRACKING

5.1 Background

Transverse cracking is a common failure that is observed as cracks that are nearly straight across the pavement, perpendicular to the direction of traffic. These cracks are mainly caused by stresses produced by temperature excursion (thermal stresses). The stress is believed to be produced by a single low-temperature excursion which, when causes the thermal shrinkage stresses to exceed the tensile strength of the pavement material, a crack will appear.

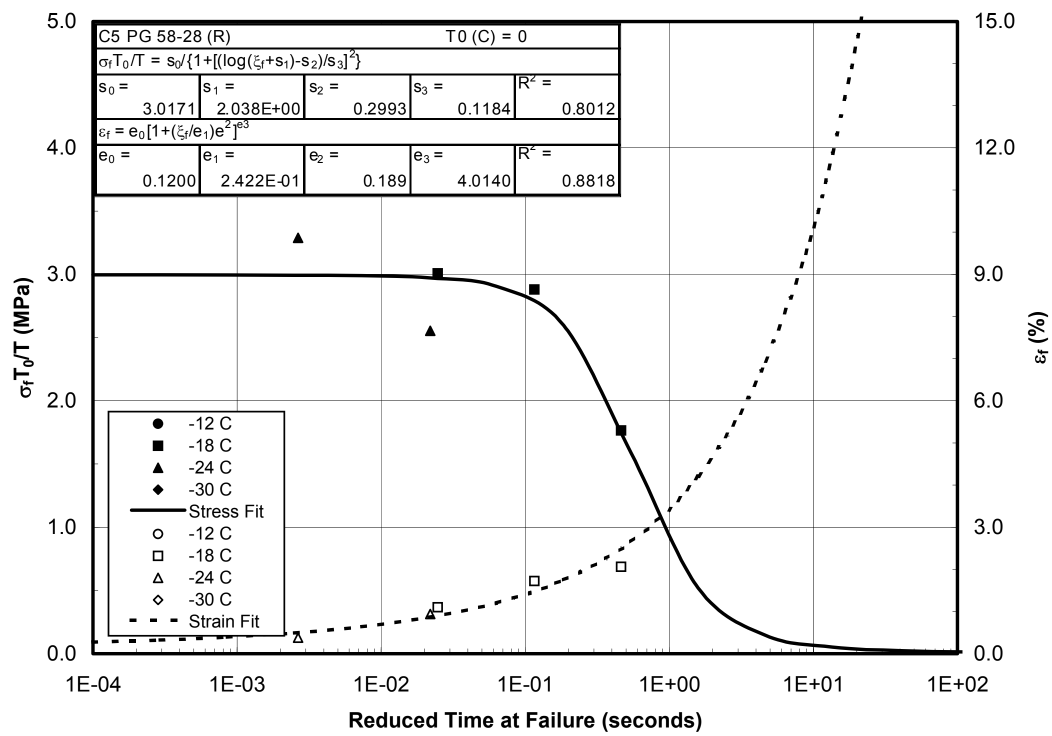
In this study, the failure properties and related thermal properties of the binders included in this study (commonly used in the state of Wisconsin) were determined by using laboratory testing. Based on these properties, the thermal cracking temperatures were estimated. The direct tension tests (DTT) were conducted in order to measure strength and strain tolerance of the binders. The bending beam rheometer (BBR) was used to acquire the rheological property of the binders as recommended by the Superpave system. In addition to these tests, the glass transition tests (GTT) were also conducted to evaluate the thermal coefficients of contraction and the glass transition temperature. The GTT test is a testing system for measuring the thermo-volumetric properties of asphalt binders allowing measuring change in volume with temperature. The ultimate purpose of this study is to establish the PG binder selection guidelines to enhance contribution of binders to resistance of cracking in the field.

5.2 Theory and Experimental Plan

The failure properties of the binders were measured using the latest direct tension test procedure as specified in the AASHTO TP3-98 and the follow up procedure explained in MP1a. The procedure was however expanded to include three strain rates at each of the three temperatures. Figure 5.1 presents an example of the data generated for the failure properties. The data represent failure stress and failure strain that are shifted along the loading time scale to produce failure master curves. The shifting is done with the time-temperature shift factors estimated from the bending beam rheometer. These master curves are necessary to estimate the critical cracking temperature based on the stress at failure and the strain at failure.

This approach, which is compatible with the recommendations of the Binder Expert Task Group, was also extended such that the analysis includes the strain at failure criterion, which is not considered in

AASHTO MP1a. The approach also considers the effect of cooling rates and solves the problem of matching the rate of cooling in the development of stress build-up to that in estimating strength or strain at failure. In the analysis procedure, the thermo-volumetric properties of binders including the glass-transition temperature are needed. Instead of making assumptions about these properties, as is recommended in AASHTO MP1a, the glass transition testing device was used to measure these properties and incorporate



them in the analysis.

Figure 5.1 Failure Stress and Strain Data and Master Curves for C5 Binder

The dilatometric glass-transition temperature (T_g) was measured using a testing procedure employed originally in the SHRP A-002A project. The T_g measurements include the change in volume as a function of temperature between +40 and -76°C. Figure 5.2 shows an example of the glass-transition measurements conducted in this project and the curve fitting , which was used to estimate the parameters.

A computer program was developed to better control the glass-transition device, to fit the data, and to calculate the parameters. In the program, the following model is used to estimate α_l, the coefficient of contraction above the glass-transition temperature; the glass-transition temperature, T_g, and α_g, the coefficient below T_g:

$$v = c_v + \alpha_g (T - T_g) + R(\alpha_l - \alpha_g) \cdot \ln\{1 + \exp[(T - T_g)/R]\}$$

Here, v is the specific volume change, c_v is a constant, and R is a regression constant related to the rate of the volume change at and near the glass-transition temperature, T_g. This model was used during the SHRP program for the same purpose. As shown in Figure 5.2, the system collects the data from two samples simultaneously, and the data from both replicates are used to fit the model. The starting temperature is 40°C, and the minimum temperature is -76°C. The data generated are used to quantify the effects of modifiers on the thermal behavior of asphalt binders.

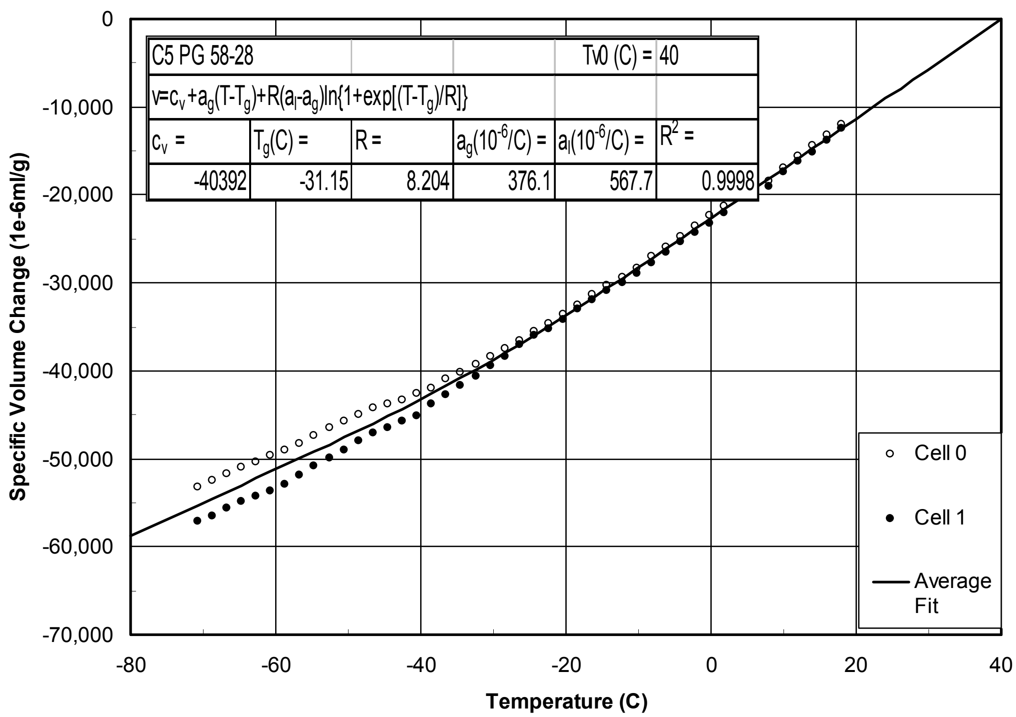


Figure 5.2 Glass Transition Measurement of C5 Binder

The low-temperature failure properties are represented by critical cracking temperatures. These are based on the concept that cracking will occur when thermal stress reaches the strength of the binder. This critical temperature was calculated with a computer program developed for this purpose. The program can calculate not only a critical temperature based on a selected conversion factor, but also a critical cracking temperature based on the concept that cracking can occur when thermal strain exceeds the failure strain. This program follows the same principles used in the procedure for determining the critical cracking temperature recommended recently by the TRB Binder Expert Task Group as included in AASHTO MP1a.

5.3 Test Results

As described earlier, the critical cracking temperatures were estimated and listed in Table 5.1. In this analysis, 12 binders in total were tested and analyzed. The program allows the calculation of the critical cracking temperature based on the concept that cracking will occur when thermal stress exceeds the strength of the binder (shown in Table 5.1 under the title 'Stress'). It can also give the critical temperature based on a selected conversion factor (shown in Table 5.1 under the title 'x18 stress' for a factor of 18). The program can also estimate the critical cracking temperature based on the concept that cracking can occur when thermal strain exceeds the failure strain (shown in Table 5.1 under the title 'Strain').

Table 5.1 Results of the Low-Temperature Cracking Tests

Binder	PG Grade	Cooling Rate (°C/hr)	Critical Cracking Temperature (°C)		
			Stress	X18 Stress	Strain
B7	PG 58-40	1	-67.4	-37.2	-47.1
D5	PG 64-40	1	-76.8	-32.9	-52.8
D4	PG 58-34	1	-62.2	-33.5	-41.8
B5	PG 64-34	1	-57.1	-15.4	-42.8
D2	PG 64-34	1	-66.8	N.A.	-47.5
C5	PG 58-28	1	-51.9	-13.6	-39.9
D1	PG 64-28	1	-54.7	-27.7	-37.3
B2	PG 64-28	1	-61.5	-20.2	-40.4
A3	PG 64-28	1	-61.4	-21.6	-41.2
B4	PG 70-28	1	-63.2	-30.5	-42.7
B3	PG 76-28	1	-59.7	-20.9	-40.9
C6	PG 76-28	1	-66.9	-27.4	-41.8
C4	PG 64-22	1	-55.9	-21.1	-36.9

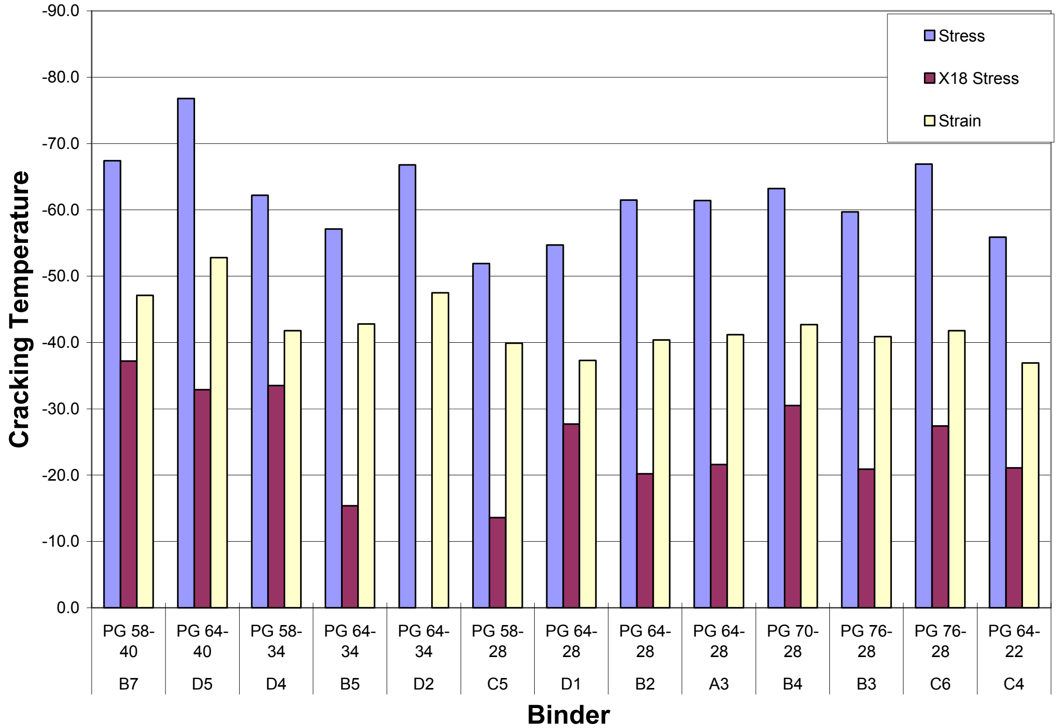


Figure 5.3 Estimated Cracking Temperatures Sorted by Criteria

Figure 5.3 illustrates the estimated cracking temperature sorted by each criterion based on the results shown in Table 5.1. From this Figure, it can be clearly seen that the cracking temperatures are highly dependent on the criterion used. The conversion factor can have an important effect that is not directly proportional to the value of the factor used and the strain criterion can give a different ranking for the binders of the same PG grade.

Table 5.1 and Figure 5.3 show that the B7 and D5 binders have better resistance to low-temperature cracking compared to the other modified binders and unmodified binder. Despite having the same low PG grade, all the tested binders of the PG xx-28 show variations in the estimated cracking temperatures; 15°C, 17°C, and 5°C for Stress, x18 stress, and Strain criteria, respectively. The unmodified binder, C5, shows the highest cracking temperatures in Stress and x18 stress criteria while another binder, C4, turns to be the lowest resistance to the cracking temperature under Strain criteria.

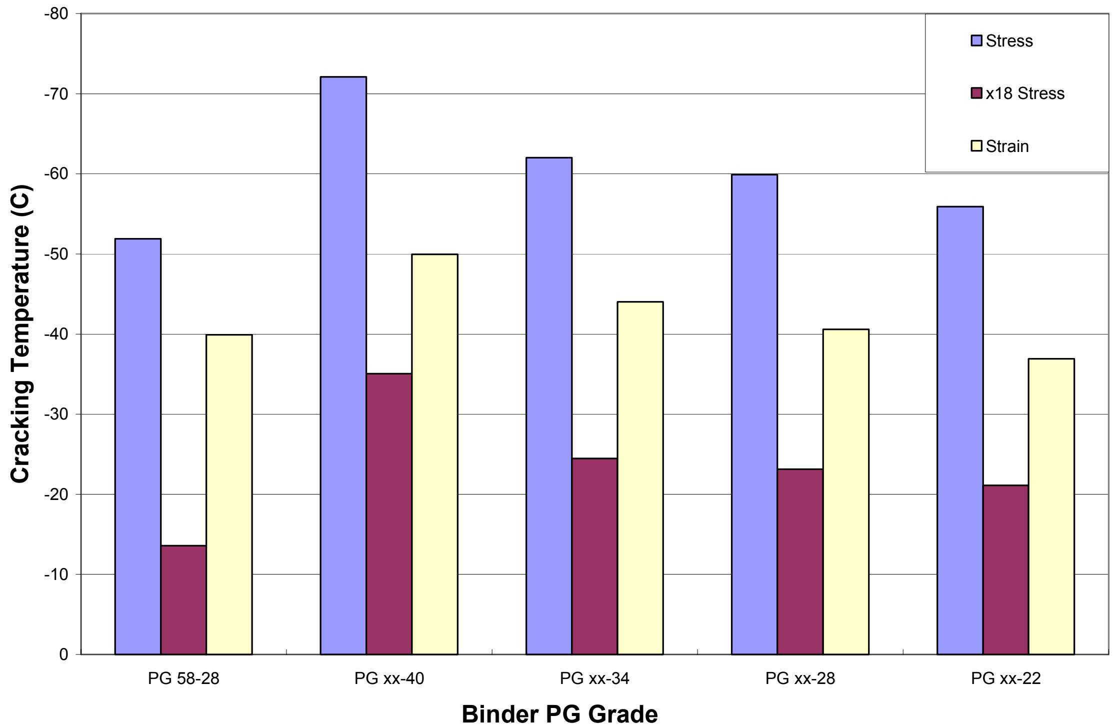


Figure 5.4 Comparison between the PG grades

The average cracking temperatures of the binders for each low PG grade are compared in Figure 5.4. The first binder is one of the PG 58-28 that is most commonly used and the rest are the averages of each PG grade. This Figure indicates that there is a significant effect of the grade on the critical cracking temperature; that is, the binders of the lowest PG grade show the lowest cracking temperatures while the highest cracking temperatures are observed from the binders of the high PG grade. And this clear distinction between the PG grades could allow constructing the linear relation between the grade and the cracking temperature, which is used to establish the binder selection criteria for low-temperature cracking.

In addition to the estimation of the low cracking temperatures, the effect of modification on binder properties was evaluated in terms of the glass transition behavior. These results are generated to be used in estimating the critical cracking temperatures. Table 5.2 shows the glass transition properties of all the tested binders. The range in the glass transition temperatures is between a low value of -55.8°C for D5 (PG 64-40 with SB modifier) and a high value of -21.3 °C for D4 (PG 58-34 with SB modifier).

Table 5.2 Summary of Glass Transition Measurements

Binder	PG Grade	T_g (°C)	$a_g(10^{-6}/°C)$	$a_l(10^{-6}/°C)$
C5	PG 58-28	-45.8	125.8	599.3
B3	PG 76-28	-24.0	427.5	574.4
A3	PG 64-28	-31.2	376.1	567.7
B2	PG 64-28	-42.4	234.7	504.4
D1	PG 64-28	-29.5	291.8	550.4
C4	PG 64-22	-27.2	345.8	527.6
B5	PG 64-34	-46.9	399.8	568.6
D2	PG 64-34	-32.4	316.1	577.5
B8	PG 70-34	-37.1	273.5	551.8
A1	PG 70-28	-25.6	329.5	596.4
B4	PG 70-28	-28.2	348.4	535.4
C2	PG 70-28	-26.7	424.1	531.7
B6	PG 76-34	-34.6	281.2	473.2
B7	PG 58-40	-53.9	258.8	574.5
B9	PG 58-34	-42.4	411.0	604.8
D4	PG 58-34	-21.3	389.5	548.7
C6	PG 76-28	-28.7	369.8	577.3
D5	PG 64-40	-55.8	326.1	566.1

As expected, two binders of the PG xx-40 grade show the lowest level in Tg, while on average the binders of the PG xx-28 have the high Tg values. Intuitively, the binders between these two grades show the medium level of Tg values.

However, such Tg values within the same PG grade reveal the significant difference of one binder to the other. For example, Tg values for the binders of the PG xx-28 vary ranging from –24°C to – 45.8°C. For the binders of PG xx-34, the difference in Tg is even greater than that of PG xx-28 binders, ranging from –21.3°C to –46.9°C. Therefore, it can be said that the glass transition temperatures are highly dependent on their modifier type and its grade. However, the great variation in the Tg values exists even within the same grade.

Table 5.2 also shows the coefficients of thermal contractions above and below the glass transition. It is observed that there is a significant difference in the coefficients of contraction between each binder. The coefficients below the Tg range from 125.8 $10^{-6}/°C$ for C5 to 427.5 $10^{-6}/°C$ for B3 while the coefficients above the Tg have less variation ranging from 473.2 $10^{-6}/°C$ for B6 to 604.8 $10^{-6}/°C$ for B9.

5.4 Construction of Binder Selection Criterion for Low-Temperature Cracking

As described earlier, the binder selection criterion for low-temperature cracking is derived from the relation between the estimated cracking temperatures and the PG grades. Figure 5.5 shows the linear relationship between these variables, the estimated cracking temperatures and the PG grades. Each point shown in the Figure is the average value of the cracking temperatures from the binders of the same low PG grade. Because we consider three different criteria of cracking, there are three linear relationships accordingly as shown in Figure 5.5. Among these three categories of thermal cracking, the conversion factor of ‘x 18 stress’ has been ruled out due to its relatively low correlation compared to the other criteria. Thus, two relationships have been selected as the low-temperature cracking criteria as follows:

1.07 x A + 36.0 < Low PG grade temperature of the region for ‘Stress’ analysis

1.38 x B + 28.0 < Low PG grade temperature of the region for ‘Strain’ analysis

Here, A denotes the estimated critical cracking temperature under the ‘Stress’ criterion from the tested binder while B is the estimated critical cracking temperature under the ‘Strain’ criterion. For example, the binder ‘C5’ has the estimated critical cracking temperatures of -51.9 and -39.9C for the ‘Stress’ and ‘Strain’ criteria, respectively. If these values are used as input for the low-temperature cracking temperature criteria described above, the temperatures of -15.5 and -27.1C are estimated for these two criteria. The decision for the use of this binder then can be made based on these temperatures. Since the highest low PG grade temperature is -28C, this binder is not adequate to be used in the areas of Wisconsin. Using the same procedure, every tested binder is analyzed to see whether it is useful for a certain climate condition. Table 5.3 shows each binder’s usable performance based on its critical cracking temperatures.

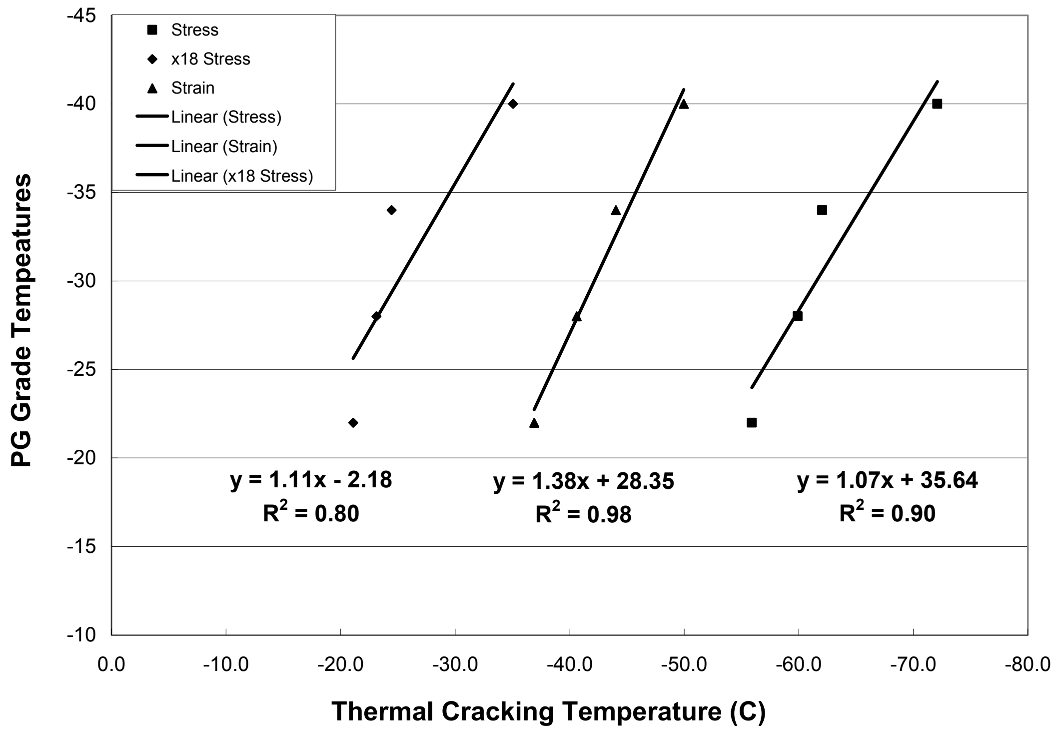


Figure 5.5 Linear Relationships between Critical Temperature and PG Grade

In Table 5.3, the 'x' denotes that the binder is applicable for the region of interest; meanwhile, the 'blank' means that the binder is not appropriate to be used. The '?' shows that the binder is questionable for use because either of critical cracking temperatures for 'Stress' and 'Strain' criteria does not meet the minimum required temperature for the area. From Table 5.3, it can be seen that some binders are applicable to the areas of interest according to their PG grade while the rest do not meet the requirement for the assigned regions based on their PG grade such as B7, D4, B5, C5, and D1. It also appears that D5 performs best among the selected binders. Meanwhile, C5 and D1 show the least performance.

Table 5.3 Applicability of Tested Binders for Low-Temperature Cracking

Binder	PG Grade	Estimated Cracking Temperature (C)		Region of PG grade		
		Stress	Strain	PG 58-28	PG 58-34	PG 52-40
B7	PG 58-40	-36.1	-37.0	x[a]	x	
D5	PG 64-40	-46.2	-44.9	x	x	x
D4	PG 58-34	-30.6	-29.7	x		
B5	PG 64-34	-25.1	-31.1	?[b]		
D2	PG 64-34	-35.5	-37.6	x	x	
C5	PG 58-28	-19.5	-27.1			
D1	PG 64-28	-22.5	-23.5			
B2	PG 64-28	-29.8	-27.8	x		
A3	PG 64-28	-29.7	-28.9	x		
B4	PG 70-28	-31.6	-30.9	x		
B3	PG 76-28	-27.9	-28.4	x		
C6	PG 76-28	-35.6	-29.7	x	?	
C4	PG 64-22	-23.8	-22.9			

[a]x denotes applicable

[b]? denotes questionable

CHAPTER 6: SUMMARY OF FINDINGS AND RECOMMENDED FUTURE WORK

6.1 Summary of Findings

The following findings are sorted by test type.

***a)* The results of the PAT** test indicate that the test is an acceptable procedure that could be used to detect particulates in asphalts. They also indicate that the binders used in Wisconsin do not contain high level of particulates. Those that contain a significant amount that is not soluble in octane, they are soluble in toluene and thus are mostly soluble or dispersible in asphalt. Since the effect of solid additives needs to be considered, it is recommended that the PAT test is used as a screening test in future specifications for asphalts in Wisconsin.

***b)* The results of the LAST test** indicates that this test is useful in studying the effects of storage stability with or without agitation. The test is however very time consuming and requires a long testing time. It is recommended that this test should be conducted by the suppliers of modified binders on samples delivered to the HMA production facility and reported to the highway agency. Since storage stability is a property of the binder and modifier, the results are not likely to change unless the modifier or binder source is changed. The test can also be used for conflict resolution when binders do not perform as expected.

***c)* Regarding the results of binder rutting,** the following points represent the summary of findings for this part of the study

- For binders with high visco-elasticity, the difference between total energy and energy dissipated in viscous flow could be very large and thus $G^*/\sin\delta$ cannot rank binder correctly with respect to rutting resistance.
- To estimate energy dissipated in viscous flow, a non-reversible cyclic loading is needed. Such loading is attained by the repeated creep testing in which an asphalt is allowed to recover and thus physically separate permanently dissipated energy from delayed elastic stored energy. The parameter Gv is proposed to evaluate the viscous component of the creep stiffness.

- It is observed that the values of Gv vary highly depending on the modification type. Because of the significant range in Gv for these binders, the concerns about the PG grading system not being able to identify better performing modified asphalts are valid.
- Modified binders can deteriorate after RTFO aging. The proposed guidelines suggest the checking of the Gv value before and after primary aging. This procedure allows rejecting materials that soften too much after aging. Although the consequences of this softening are not known in terms of pavement performance, it could mean degrading of polymers, which cannot be considered beneficial.
- The criteria proposed allow consideration of traffic speed and volume without grade bumping. The adjustment factors for these conditions are derived based on field conversion factors and mechanistic understanding of the parameters used in the criteria.
- The criteria proposed for binder rutting are based on field conversion factors derived from a limited amount of data. The limits are derived based on assumptions and approximations that may or may not be completely valid. The limits are considered as a starting point for further evaluation by the Wisconsin DOT and the Industry. It is hoped that a major field study will be initiated to collect enough data to derive more reliable field conversion factors.

d) **Regarding the results of binder fatigue** the following points give a summary of the findings for this part of the study:

- The fatigue resistance of binders, as measured by the Np20 values, shows a great disparity for binders of the same PG grade. This points out that the current PG grade testing is not a good indicator of binder contribution for fatigue resistance in asphalt mixtures. The results show that there are significant effects of temperature and pavement structure on the fatigue performance of binders.
- Considering the traffic speed by changing the input energy only is a very convenient approach. Since all the testing is done at high frequency (10 Hz), the time of testing is reduced considerably.
- The ranking of binders according to the proposed criteria agrees with trends reported for fatigue testing of asphalt mixtures as the binders show better performance for strong structure than for weak structure. Also, the binders show

better fatigue resistance when the testing frequency, which represents traffic speed, fast than when the traffic is slow.

- The proposed limits are derived based on assumptions and approximations that may or may not be completely valid. The assumption that the for the strong structure the binder perform in he linear range and for weak structure in the non linear range might not be very accurate. If a better approximation of the real stress (strain) in the pavement is known, it should be used for determining the initial input energy.

***e)* Regarding the results of binder low temperature cracking,** the following points give a summary of the findings for this part of the study:

- Building on the concept of estimating critical cracking temperatures from strength and thermal stress, as included in the AASHTO MP1a procedure, the binders in this study were tested to estimate 2 critical cracking temperatures based on strength and strain tolerance. The results show that these two temperatures are not the same for binders of the same grade. It is proposed that such a procedure could give better evaluation of the contribution of modified binders to thermal cracking of pavements.
- In this project the glass transition temperature (Tg) of binders were measured and they are found to follow a logical trend with PG low temperature grading. Lower PG grade binders show lower Tg. The Tg testing are necessary for better estimation of coefficients of thermal contraction which are used in estimating cracking temperatures.
- The results indicate that using the AASHTO MP1a procedure could result in misleading ranking of the binders. The critical cracking temperature based on the strain at failure gives better relationship to the low temperature grading. Since no field data is available yet to verify which procedure is better, new models for field conversion factors are introduced to estimate critical cracking temperatures.
- The need for the TG measurements, although clear in modeling, should be validated by a field study. The direct tension measurements collected clearly indicate that for modified binders, the PG grading is not sufficient for comparing effects of modifiers.

6.2 Recommended Future Work

This research project was intended to deliver a revision of the current PG binder selection guidelines used in Wisconsin. The revision includes advanced methods of evaluating modified binders and specific procedures for considering traffic speed, traffic volume, and pavement structure in the selection of binders other than grade shifting. The revised guidelines were based on an extensive library of performance-related properties of binders commonly used in Wisconsin. The ultimate goal of this research was to establish initial specification limits that could serve as a base for further field validation. Therefore, these tentative guidelines need field validation to ensure that they can be successfully implemented and the asphalt producers can in fact deliver the recommended grades at realistic cost. The field validation is also required to ensure that the required testing and evaluations are truly performance-related. The focus of the field validation research should be on the selection of experimental pavement sections in various regions of the state to validate the guidelines and establish a database that will allow the continuous revision and adjustment of the binder selection guideline by the WisDOT. The tentative plans for the field validation are presented in Appendix II.

The eventual implementation of the proposed guidelines involves new requirements for accepting binders. Therefore contractors, asphalt suppliers and DOT staff need to be informed and eventually trained in the new testing procedures. A training program should start with some demonstrations for the test in the University of Wisconsin Madison or at the DOT central laboratory. This could be done by University staff or by DOT staff after training by the researchers who participated in development of testing systems.

APPENDIX I: LAST TEST RESULTS

Summary of Result for LAST Test
24 Hours Sampling Time, External Heat WITHOUT Agitation (Part I)

Frequency (rad/s)	5.0		50.0		5.0		50.0	
Temperature (ºC)	HT		HT		IT		IT	
A1	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d
	1.02	1.00	1.02	1.00	1.01	0.99	1.00	0.99
	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d
	1.01	0.97	0.98	0.98	1.01	1.04	1.08	1.02
B2	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d
	1.13	0.99	1.11	0.99	1.16	0.98	1.13	0.98
	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d
	1.05	0.97	0.99	0.98	0.99	1.01	1.00	1.02
B5	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d
	1.08	0.99	1.07	0.99	1.08	0.99	1.08	1.00
	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d
	0.95	1.01	0.96	1.01	0.95	1.00	0.94	1.00
B6	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d
	1.02	1.01	1.02	1.00	1.06	0.99	1.05	0.99
	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d
	0.87	1.02	0.91	1.04	1.01	1.01	1.00	1.00
B7	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d
	0.94	1.03	0.99	1.02	0.98	1.01	0.98	1.00
	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d
	0.82	1.04	0.87	1.03	0.92	1.01	0.93	1.01
B8	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d
	1.04	1.00	1.05	1.00	1.05	1.00	1.04	1.00
	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d
	0.86	1.05	0.92	1.04	0.95	1.00	0.95	1.00
B9	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d
	1.23	0.96	1.15	0.97	1.17	0.98	1.14	0.98
	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d
	0.98	1.04	1.04	1.02	1.05	0.99	1.02	0.99

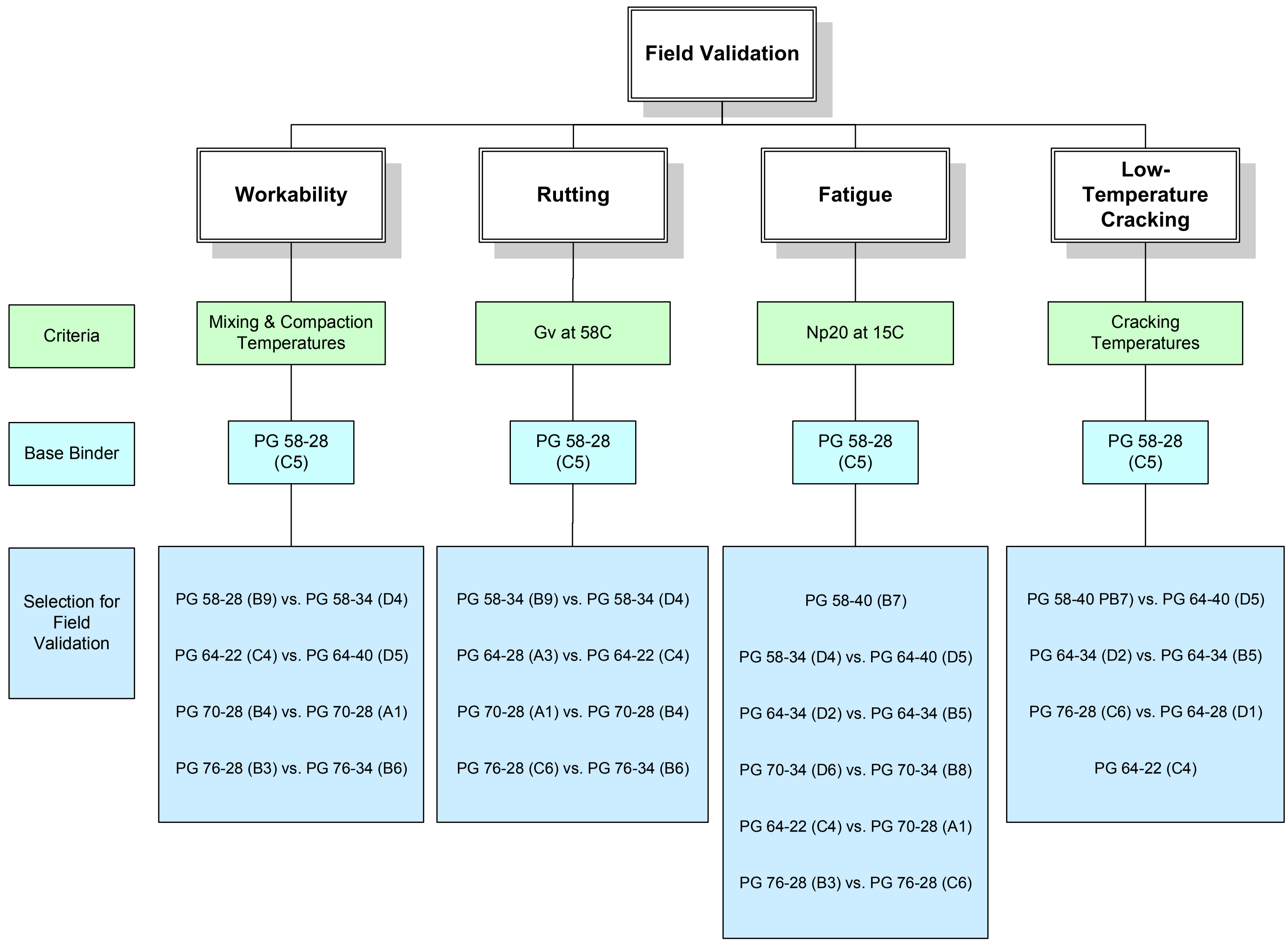
Summary of Result for LAST Test
24 Hours Sampling Time, External Heat WITHOUT Agitation (Part II)

Frequency (rad/s)	5.0		50.0		5.0		50.0	
Temperature (ºC)	HT		HT		IT		IT	
C2	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d
	1.01	1.01	1.02	1.00	1.02	1.00	1.03	1.01
	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d
	0.98	0.99	0.97	0.99	0.89	1.00	0.90	1.01
C5	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d
	0.89	1.00	0.89	1.00	0.84	1.05	0.89	1.10
	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d
	1.03	1.00	1.02	1.00	0.95	1.00	0.87	1.01
C6	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d
	0.94	0.95	0.91	0.99	1.00	1.01	1.03	1.02
	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d
	1.06	0.91	0.98	0.96	0.78	1.00	0.80	1.01
D2	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d
	1.10	0.99	1.08	0.99	1.07	0.99	1.05	0.99
	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d
	1.03	0.99	1.02	0.99	1.00	1.00	1.00	1.00
D4	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d
	0.98	1.00	0.99	1.00	0.94	1.00	0.95	1.00
	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d
	1.32	0.98	1.26	0.98	1.34	0.97	1.25	0.97
D5	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d
	0.65	0.73	0.43	0.77	0.09	1.01	0.10	1.14
	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d
	1.16	0.99	1.24	1.01	2.80	1.06	2.90	1.07
D6	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d
	0.97	1.03	0.99	1.01	0.33	0.85	0.10	0.87
	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d
	1.09	0.92	0.99	0.95	0.64	0.97	0.58	0.98

Summary of Result for LAST Test
24 Hours Sampling Time, External Heat WITH Agitation

Frequency (rad/s)	5.0		50.0		5.0		50.0	
Temperature (ºC)	HT		HT		IT		IT	
A1	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d
	1.00	1.05	1.04	1.02	1.08	1.00	1.08	1.00
	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d
	1.20	0.98	1.12	0.95	1.18	1.05	1.26	1.02
A3	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d
	1.03	0.99	1.02	1.00	1.03	1.00	1.01	0.99
	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d
	1.11	1.01	1.13	1.00	1.15	1.00	1.13	0.99
B2	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d
	1.05	0.99	1.03	1.00	1.00	1.00	1.01	1.00
	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d
	1.28	0.98	1.23	0.98	1.24	0.98	1.18	0.98
B3	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d
	1.31	0.95	1.23	0.97	1.24	0.97	1.17	0.97
	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d
	0.58	1.08	0.64	1.06	0.70	1.07	0.77	1.07
B7	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d
	1.03	0.99	1.03	1.00	1.04	1.00	1.03	0.99
	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d
	1.27	0.95	1.19	0.97	1.16	0.99	1.15	0.99
B8	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d
	0.98	1.00	0.98	1.00	0.99	1.00	0.99	1.00
	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d
	1.28	0.95	1.21	0.97	1.21	0.99	1.19	0.99
B9	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d
	1.03	0.99	1.02	1.00	1.03	1.00	1.03	1.00
	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d
	1.11	0.99	1.10	0.99	1.11	0.99	1.08	0.99
D1	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d
	0.96	1.00	0.96	1.00	0.95	1.00	0.98	1.01
	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d
	0.93	1.01	0.95	1.01	1.01	1.02	1.02	1.01
D2	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d
	1.03	1.00	1.03	1.00	1.01	1.00	1.00	1.00
	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d
	0.98	1.00	0.98	1.00	0.98	1.00	0.98	1.00
D5	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d	Rs G*	Rs d
	1.05	0.99	1.04	0.99	1.06	1.01	1.08	1.01
	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d	Rd G*	Rd d
	1.05	0.97	1.02	0.97	1.00	0.43	1.14	1.11

APPENDIX II: TEST PLANS FOR FIELD VALIDATION


Field Validation
Workability
Rutting
Fatigue
Low-Temperature Cracking
Criteria
Mixing & Compaction Temperatures
Gv at 58C
Np20 at 15C
Cracking Temperatures
Base Binder
PG 58-28 (C5)
PG 58-28 (C5)
PG 58-28 (C5)
PG 58-28 (C5)
Selection for Field Validation
PG 58-28 (B9) vs. PG 58-34 (D4)
PG 64-22 (C4) vs. PG 64-40 (D5)
PG 70-28 (B4) vs. PG 70-28 (A1)
PG 76-28 (B3) vs. PG 76-34 (B6)
PG 58-34 (B9) vs. PG 58-34 (D4)
PG 64-28 (A3) vs. PG 64-22 (C4)
PG 70-28 (A1) vs. PG 70-28 (B4)
PG 76-28 (C6) vs. PG 76-34 (B6)
PG 58-40 (B7)
PG 58-34 (D4) vs. PG 64-40 (D5)
PG 64-34 (D2) vs. PG 64-34 (B5)
PG 70-34 (D6) vs. PG 70-34 (B8)
PG 64-22 (C4) vs. PG 70-28 (A1)
PG 76-28 (B3) vs. PG 76-28 (C6)
PG 58-40 PB7) vs. PG 64-40 (D5)
PG 64-34 (D2) vs. PG 64-34 (B5)
PG 76-28 (C6) vs. PG 64-28 (D1)
PG 64-22 (C4)